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MFN 09-772 Revision 1

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Subject: **Revised Response to Portion of NRC RAI Letter No. 368 Related to ESBWR Design Certification Application - DCD Tier 2 Section 3.8 - Seismic Category I Structures; RAI Number 3.8-96 S05 Revision 1**

The purpose of this letter is to submit the GE Hitachi Nuclear Energy (GEH) revised response to a portion of the U.S. Nuclear Regulatory Commission (NRC) Request for Additional Information (RAI) letter number 368 sent by NRC letter dated September 10, 2009 (Reference 1). GEH initially responded to RAI 3.8-96 S05 Revision 1 in Reference 2. This letter transmits a revsion to GEH's response to RAI 3.8-96 S05 Revision 1 after interactions with the staff to clarify Reference 2.

Enclosure 1 contains GEH's revised response to RAI Number 3.8-96 S05 Revision 1. Revision bars in the right hand column and text strike throughs identify the changes to Reference 2. Revised DCD Markups are found in Enclosures 2 as a result of GEH's revised response to RAI 3.8-96 S05 Revision 1.

All remaining pages in Reference 2 Enclosures and Attachments not revised by this letter remain valid.

If you have any questions or require additional information, please contact me.

Sincerely,

Richard E. Kingston

Richard E. Kingston
Vice President, ESBWR Licensing

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NRO

References:

1. MFN 09-598 Letter from U.S. Nuclear Regulatory Commission to J. G. Head, GEH, *Request For Additional Information Letter No. 368 Related to ESBWR Design Certification Application* dated September 10, 2009
2. MFN 09-772 Letter from R. E. Kingston, GEH, to the U.S. Nuclear Regulatory Commission *Response to Portion of NRC RAI Letter No. 368 Related to ESBWR Design Certification Application - DCD Tier 2 Section 3.8 - Seismic Category I Structures; RAI Number 3.8-96 S05* dated December 12, 2009

Enclosures:

1. Revised Response to Portion of NRC RAI Letter No. 368 Related to ESBWR Design Certification Application - DCD Tier 2 Section 3.8 – Seismic Category I Structures; RAI Number 3.8-96 S05, Revision 1
2. Revised Response to Portion of NRC RAI Letter No. 368 Related to ESBWR Design Certification Application - DCD Tier 2 Section 3.8 – Seismic Category I Structures; DCD Tier 1 and Tier 2 Markups for RAI Number 3.8-96 S05 Revision 1

cc:	AE Cubbage	USNRC (with enclosures)
	JG Head	GEH/Wilmington (with enclosures)
	DH Hinds	GEH/Wilmington (with enclosures)
	HA Upton	GEH/ San Jose (with enclosures)
	eDRFSection	0000-0107-6984 R1(RAI 3.8-96 S05 R1)

ENCLOSURE 1

MFN 09-772 Revision 1

Revised Response to NRC RAI Letter No. 386

Related to ESBWR Design Certification Application[1]

DCD Tier 2 Section 3.8 – Seismic Category I Structures

RAI Number 3.8-96 S05, Revision 1

[1] Original Response, Supplement 1, Supplement 2, Supplement 3 and Supplement 4 previously submitted under MFNs 06-407; 06-407, Supplement 2; 06-407, Supplement 3; 06-407, Supplement 14 and 09-449 without DCD updates are included to provide historical continuity during review.

NRC RAI 3.8-96

DCD Section 3.8.5.5 presents two specifications of appropriate safety factors (SF) for foundation design. The SF against sliding indicates that sliding resistance is judged as the sum of both shear friction along the basemat and passive pressures induced due to embedment effects. However, the DCD does not indicate (1) how these effects are to consider consistent lateral displacement criteria (that is, the displacement effect on passive pressure is not the same as on friction development) and (2) how the effect of waterproofing is to impact the development of basemat friction capacity. DCD Section 3.8.5.5 needs to clearly indicate how these effects are incorporated into the standard plant design for the considered range of acceptable site conditions considered.

Include this information in DCD Section 3.8.5.5. In addition, (1) identify the applicable detailed report/calculation (number, title, revision and date, and brief description of content) that will be available for audit by the staff, and (2) reference this report/calculation in the DCD.

GE Response

a) As stated in the response to NRC RAI 3.7-35, SASSI analyses were performed to address the embedment effect. It was confirmed that the base shears calculated by the SASSI analyses, which consider the embedment effect, are less than those obtained by design seismic analyses that neglect the embedment effect. The use of higher base shears calculated without the beneficial effect of embedment is deemed conservative for the sliding evaluation without explicit consideration of consistent lateral displacement criteria for passive pressure and friction resistance.

b) Please see NRC RAI 3.8-89 for the response to impact of waterproofing.

(1) The applicable detailed reports/calculations that will be available for the NRC audit are:

26A6652, *RB FB Stability Analysis Report, Revision* 2, April 2006, which contains the stability calculations of the Reactor Building/Fuel Building.

26A6654, *CB Stability Analysis Report, Revision* 2, April 2006, which contains the stability calculations of the Control Building.

(2) Since this information exists as part of GE's internal tracking system, it is not necessary to add it to the DCD.

No DCD change will be made in response to this RAI.

NRC RAI 3.8-96, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

GE needs to clarify the response to this RAI and revise Section 3.8.5.5 to be consistent with their response. Does GE calculate the SF against sliding by only considering the basemat shear friction? If not, GE needs to better explain the method used in the light of the question asked. GE also needs to explain (1) Do the exterior walls need to be designed for passive pressures as implied in the last sentence of item (a) of the response? (2) Are both base shear and passive pressures being relied upon for lateral restraint? (3) the friction coefficient used in the analysis and its technical bases, (4) how lift-off effects are captured in the sliding analysis, (5) the capacity of the mud mat to resist applied loads, and (6) what effect the use of chemical crystalline powder in the mud mat has on the assumed structural properties. Potential leaching of the mud mat due to groundwater is being reviewed under RAI 3.8-81.

During the audit, GE indicated the following:

(1) & (2) GE explained the answer to both is yes. The seismic stick model did not consider embedment effects while the stability calculations (soil sliding), using this shear force, did consider soil friction and soil passive pressure. However, the SASSI did consider soil embedment and it was shown that the resulting shear loads are smaller than those calculated by the seismic stick model. GE indicated that they will determine an appropriate method to consider the seismic shear force from the seismic stick model and/or SASSI analysis in their calculation of sliding stability calculation. The method used will ensure consistency of the deformation in developing the frictional soil resistance and soil passive pressure. Also, the design of the foundation walls will consider the appropriate pressures from the SASSI analysis and passive soil pressures used in the sliding stability calculations.

(3) GE will provide the reference for the static and dynamic coefficient of friction values. This would be needed if GE is not able to show that the soil frictional resistance alone can resist the seismic shear force.

(4) GE will provide additional justification to demonstrate that the effects of uplift are not significant.

(5) GE will expand on the description of the mud mat and provide the minimum applicable requirements (e.g., ACI Code).

(6) GE explained that this material has no deleterious effect on the concrete and has been used and approved at other NPPs.

GE Response

(1) & (2) Table 3.8-96(1) summarizes the evaluation results of the foundation sliding analyses for generic site conditions.

The seismic loads used in the evaluation are obtained by seismic response analysis using the lumped soil spring stick model (DAC3N analyses). Since the lumped soil spring model does not consider embedment effects, the resulting shear loads are larger than those calculated by SASSI analyses. The use of higher base shear is conservative for the foundation stability evaluation.

Sliding resistance is composed of the following:

- Friction force at the basemat bottom surface
- Cohesion force at the basemat bottom surface
- Passive soil pressure at the basemat side surface
  For the RB/FB and CB, the gap between the building and excavated soil is filled with concrete up to the top level of the basemat or higher. Since the basemat is constrained by rigid concrete backfill, the passive soil pressure is mobilized for the region.
- Passive soil pressure on walls
  The passive soil pressures considered are the envelope lateral soil pressures obtained from the elastic solution based on ASCE 4-98, Section 3.5.3.2 and SASSI analysis results, which are used in the wall design.

(3) Only the static coefficient of friction is used for stability evaluation. Coefficient of friction, μ, is calculated by the following equation.

$$\mu = \min(\tan\phi, 0.75)$$

where,

ϕ = Angle of internal friction (30° for soft and medium soil, 40° for hard soil).

The minimum angle of internal friction will be specified to be 30° in DCD Tier 2 Table 2.0-1 as a site requirement.

(4) Sliding resistance is composed of passive soil pressure, friction and cohesion forces at the basemat bottom. Uplift of the basemat has no effect on the passive soil pressure. The friction force at the basemat bottom is also not influenced by the uplift, because the friction force is calculated by (normal compressive force) x (friction coefficient). Because the basemat uplift has no effect on both the normal compressive force and friction coefficient, the resulting friction force is unchanged even if uplift occurs. As for the cohesion force, since it is calculated by (cohesion stress) x (contact area of basemat), the value is reduced if the basemat is uplifted. However, the contribution of the cohesion force to the total resistance is relatively small as shown in Table 3.8-96(1). The reduction of the cohesion force due to uplift has little impact on the total resistance.

(5) The mud mat construction is performed in accordance with the same standards and requirements as the basemat to avoid possibility of errors in the field.

(6) The crystalline powder used is the same material approved for use in AP-1000 and has no deleterious effect on concrete. It forms a substantial waterproofing barrier to prevent water infiltration or ex-filtration.

Table 3.8-96(1) Sliding Evaluation Results

(i) RBFB

Building width X	70.0 m					
Building width Y	49.0 m					
Total Weight	2360 MN					
Buoyancy	652 MN					
Soil Condition	Soft		Medium		Hard	
Vertical Seismic Load	676 MN		1159 MN		1103 MN	
Minimum Vertical Load	1438 MN		1244 MN		1267 MN	
	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir
Fv: Horizontal Seismic Force (MN)	*899*	*787*	*1462*	*1619*	*1486*	*1243*
Fub: Bottom Friction Force (MN)	830	830	718	718	950	950
Fc: Effective Cohesion Force (MN)	0	0	343	343	1166	1166
Fpb: Passive Pressure for Basemat (MN)	132	188	213	304	539	769
Fdsf: Passive Soil Pressure on Wall (MN)	440	644	440	644	440	644
Fr: Sliding Resistance (=Fub+Fc+Fpb+Fdsf)	*1402*	*1663*	*1714*	*2010*	*3095*	*3530*
FS (=Fr/Fv)	**1.56**	**2.11**	**1.17**	**1.24**	**2.08**	**2.84**

(ii) CB

Building width X	30.3 m					
Building width Y	23.8 m					
Total Weight	173 MN					
Buoyancy	101 MN					
Soil Condition	Soft		Medium		Hard	
Vertical Seismic Load	72 MN		79 MN		100 MN	
Minimum Vertical Load	43 MN		40 MN		32 MN	
	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir
Fv: Horizontal Seismic Force (MN)	*105*	*100*	*97*	*94*	*101*	*91*
Fub: Bottom Friction Force (MN)	25	25	23	23	24	24
Fc: Effective Cohesion Force (MN)	0	0	72	72	245	245
Fpb: Passive Pressure for Basemat (MN)	36	46	64	82	173	220
Fds: Passive Soil Pressure on Wall (MN)	58	74	58	74	58	74
Fr: Sliding Resistance (=Fub+Fc+Fpb+Fds)	*119*	*145*	*218*	*251*	*500*	*563*
FS (=Fr/Fv)	**1.13**	**1.44**	**2.23**	**2.67**	**4.94**	**6.22**

Note:

1. Minimum vertical load: Wm = Wt – Fb – 0.4Fa
   where,
   Fb: Buoyancy due to groundwater
   Fa: Vertical seismic force
2. Bottom friction force: Fub = Wm* μ
   where,
   μ: friction coefficient
3. Fv and Fa are obtained by seismic lumped soil spring stick model analyses (DAC3N analyses)

DCD Tier 2 Table 2.0-1, Subsections 3G.1.5.5 and 3G.2.5.5 and Tables 3G.1-57 and 3G.2-26 have been revised. DCD Tier 2 Figures 3G.1-65 and 3G.2-15 have been added. The pages (pp. 2.0-3, 3G-16, 3G-123, 3G-189, 3G-194, 3G-215 & 3G-230) revised in DCD Tier 2 Revision 3 for this response are attached.

DCD Impact

As stated above.

NRC RAI 3.8-96, Supplement 2

NRC Assessment from Chandu Patel E-mail Dated May 24, 2007

The applicant has not used a consistent set of criteria to determine the safety factor against sliding and also needs to provide the technical bases for some of the parameters used in the analysis results that are presented. The staff requests the applicant to address the following:

(1) The fourth bullet in the list of items that comprise the sliding resistance is identified as "passive soil pressure on walls." This terminology is misleading since the information included under this item is the elastic lateral soil pressure. If passive soil pressures are being credited to provide sliding resistance, explain how these pressures are calculated and confirm that the walls are designed to resist these forces. If elastic lateral soil pressures on the walls are being credited to provide sliding resistance, it is not consistent to use these elastic soil pressures with the passive soil pressures at the basemat side surface. Also, explain how the passive soil pressures are calculated for the basemat side surface.

(2) Passive soil pressure at the basemat side surface is being credited to provide sliding resistance, which means that the static friction resistance at the bottom of the basemat is overcome. Therefore, explain why a dynamic coefficient of friction is not used to calculate the friction force at the basemat bottom surface.

(3) How has GE determined that there are sufficient soil sites that would have an angle of internal friction of 30 degrees or greater? What would a COL applicant be required to do if a site has a soil friction angle of less than 30 degrees?

(4) Provide a description of the formulations used to calculate the cohesion resisting forces and discuss how the material properties were determined for the analysis.

(5) Provide the technical basis for assuming that medium soils with an angle of internal friction of 30 degrees would also have the effective cohesion resisting forces reported in the analysis results in Table 3.8-96(1). Why is the cohesion value in Table 3.8-96(1) equal to zero for soft soils?

(6) Provide the technical basis for assuming that the hard soil/rock conditions have the effective cohesion resisting forces reported in the analysis results in Table 3.8-96(1).

(7) Why does the response indicate that the cohesion force contribution to total force is small when Table 3.8-96(1) shows that it is quite large for hard soils? For the RBFB medium soil condition, a small change in the cohesion force could result in a factor of safety of less than 1.1. In the light of these observations, further justification is needed to support the statement that the reduction of the cohesion due to uplift has little impact on the total resistance.

(8) Describe the COL requirements for the backfill material for the gap shown in Figures 3G.1-65 and 3G.2-15. Will the backfill material be required to have a stiffness defined by its shear wave velocity which is at least equal to the shear wave velocity of the surrounding insitu soil? If not, explain why not. Also, clarify that the backfill material will completely fill the gap above the concrete backfill to the grade level.

(9) The note in Table 3.8-96(1) implies that the 100-40-40 three directional combination method was used for the sliding evaluation. The data in the tables above the note, however indicate that a two dimensional (one horizontal and one vertical) check was made for calculating the factor of safety. In this evaluation the bottom friction force is derived based on the total vertical load consisting of dead weight minus the buoyancy effect minus 0.40 times the vertical seismic force. Since a simplified two dimensional approach (i.e., N-S & Vertical and then E-W & Vertical) is being used to demonstrate the factors of safety against sliding and overturning, the 100-40-40 rule is not considered to be appropriate. The typical approach that is utilized for checking sliding and overturning in accordance with the SRP 3.8.5 requirements is to use the dead load minus the buoyancy effect and then subtract the full vertical seismic load for the N-S & Vertical check and the E-W & Vertical check. If any other method is utilized, then GE needs to provide the technical justification for the approach. Note that 90% of the dead load (including the buoyancy effect) should be utilized as specified in Note 1 of DCD Table 3.8-15, which is also in accordance with ACI 349 requirements.

GEH Response

(1) In the calculations shown in Table 3.8-96(1), elastic lateral soil pressures on the walls were credited to provide sliding resistance. This is conservative for sliding evaluation since actual passive pressures, if mobilized, would be higher. Wall design is based on elastic lateral soil pressures. As discussed in the response to Item (4), the required factor of safety can be satisfied without considering the sliding resistance from the elastic lateral soil pressures. Passive pressure is mobilized on the side surface of the basemat since the basemat is constrained by rigid concrete backfill. The passive pressure at the basemat side is calculated using the following equations:

$$P_p = k_p \gamma' H + \gamma_w H_w + k_p q + 2C\sqrt{k_p}$$

$$k_p = \frac{1 + \sin\phi}{1 - \sin\phi}$$

where,

k_p = Passive pressure coefficient
H = Height of soil column
H_w = Height of water column
γ' = Effective weight of soil. Use buoyant unit weight below water table and moist unit weight above water table.
γ_w = Unit weight of water

q = Magnitude of surcharge load per unit area
ϕ = Angle of internal friction of soil
C = Cohesion

The stress in the basemat generated by passive soil pressures is 2.45 MPa for the Hard site condition and is less than 10% of the concrete compressive strength. The stress is acceptable for the basemat design.

(2) The shear strength of soil, i.e., the resistance at the basemat bottom, is composed of friction and cohesion. It is generally recognized that the strength of soil for dynamic loads is larger than that for static loads. Therefore, calculations using static coefficient of friction, i.e., calculations based on the static strengths, are conservative.

(3) Table 2-6 from Reference 1 shows that a 30° angle of internal friction is a reasonable lower bound for competent soil material. A site-specific sliding evaluation would be performed if the angle of friction of the site-specific foundation material is lower than 30°. In DCD Tier 2 Subsection 2.0-1-A, the COL applicant referencing the ESBWR DCD is required to demonstrate that the site characteristics, which includes angle of internal friction, of a given site fall within ESBWR DCD design parameter values shown in DCD Tier 2 Table 2.0-1.

TABLE 2-6 Representative values for angle of internal friction ϕ

	Type of test*		
Soil	Unconsolidated-undrained U	Consolidated-undrained CU	Consolidated-drained CD
Gravel			
Medium size	40–55°		40–55°
Sandy	35–50°		35–50°
Sand			
Loose dry	28-34°		
Loose saturated	28–34°		
Dense dry	35–46°		43–50°
Dense saturated	1–2° less than dense dry		43–50°
Silt or silty sand			
Loose	20–22°		27–30°
Dense	25–30°		30–35°
Clay	0° if saturated	3–20°	20–42°

* See a laboratory manual on soil testing for a complete description of these tests, e.g., Bowles (1986*b*).

Notes:

1. Use larger values as γ increases
2. Use larger values for more angular particles
3. Use larger values for well-graded sand and gravel mixtures (GW, SW)
4. Average values for
   Gravels: 35–38°
   Sands: 32-34°

(4) In Reference 1 it is stated that the ultimate bearing capacity, q_u, can be nine times cohesion, c. In the same reference, it is suggested to use 0.5 to 0.7 of c for sliding stability evaluations. That is, the cohesion used for sliding evaluations, c', can be evaluated by the following equation as a function of the ultimate bearing capacity:

$$c' = 0.5 \times q_u / 9 = q_u / 18$$

The expected ultimate bearing capacities of the ESBWR design need to be larger than the maximum soil bearing stresses summarized in the DCD Tier 2 Table 3G.1-58 for the RBFB and Table 3G.2-27 for the CB, respectively. These are the demand pressures.

Assuming the demand pressures are the actual ultimate bearing capacities, the associated cohesions can be conservatively evaluated by substituting the maximum soil bearing stresses into q_u in the above equation. The resulting cohesions are summarized in Table 3.8-96(2). The sliding stability evaluations were updated using these cohesions. The results are shown in Table 3.8-96(3). The calculated factors of safety (FS) satisfy the allowable value of 1.1. In DCD Tier 2 Revision 4, Tables 3G.1-57 and 3G.2-26 were revised in accordance with the results in Table 3.8-96(3). The revised pages 3G-123 and 3G-228 in DCD Tier 2 Revision 4 are attached.

In the calculations in Table 3.8-96(3), the elastic lateral soil pressures on the walls discussed in Item (1) above are conservatively neglected. The passive pressure utilized is only at the basemat side as described Item (1) above.

(5) See response to Item (4) where cohesion is taken to be a function of the ultimate bearing capacity.

(6) See response to Item (4) where cohesion is taken to be a function of the ultimate bearing capacity.

(7) According to the basemat uplift analysis results, which are shown in the DCD Tier 2 Figures 3G.1-60 and 3G.1-61, the ratios of contact area of the basemat are about 80% and 85% for N-S and E-W directions, respectively. Since the cohesion is effective at the contact area only, it is reduced in proportion to the ratio of contact area. The FS listed in Table 3.8-96(3) have sufficient margins for the reduced contact area of 80%.

(8) The shear wave velocity of the backfill material is not required to be at least equal to that of the surrounding in situ soil. This is because lateral soil/backfill was neglected in the design basis seismic analysis using the lumped-mass soil spring approach (DCD Tier 2 Subsection 3A.5.1). This approach was confirmed to be conservative as compared to the results of the SASSI analysis taking into account embedment (DCD Tier 2 Subsection 3A.8.7). The gap is completely filled with compacted engineered backfill material. This statement is included in notes to DCD Tier 2 Revision 4 Figures 3G.1-65 and 3G.2-17. The revised pages 3G-189 and 3G-245 in DCD Tier 2 Revision 4 are attached.

(9) Alternate sliding stability is performed for the three dimensional seismic loads in accordance with the 100-40-40 rule.

Applied horizontal seismic forces and sliding resistances are schematically shown in Figure 3.8-96(1). Among the resistances, the basemat bottom friction and cohesion act in the direction of the resultant seismic force and their magnitudes are the same as those in the 2-dimensional evaluation.

Resistances due to the passive soil pressures applied to the basemat side surfaces are evaluated as follows:

Soil pressures are applied perpendicular to the basemat. The component in the direction of the seismic force is calculated by the following equation:

$$F = F_x \cos\theta + F_y \sin\theta \qquad (1)$$

From the equilibrium of forces in the direction perpendicular to the seismic forces, the following equation needs to be satisfied:

$$F_x \sin\theta = F_y \cos\theta \qquad (2)$$

By substituting Eq. (2) into Eq. (1), the following equations are obtained:

$$F_1 = \left(\cos\theta + \frac{\sin^2\theta}{\cos\theta}\right)F_x = \left(\cos\theta + \frac{1-\cos^2\theta}{\cos\theta}\right)F_x = \frac{F_x}{\cos\theta} \qquad (3a)$$

or

$$F_2 = \left(\sin\theta + \frac{\cos^2\theta}{\sin\theta}\right)F_y = \left(\sin\theta + \frac{1-\sin^2\theta}{\sin\theta}\right)F_y = \frac{F_y}{\sin\theta} \qquad (3b)$$

F_1 and F_2 reach their maximum values when F_x and F_y are equal to the resultant forces due to passive soil pressures. As a result, the resistance due to passive soil pressures is obtained by the following equations:

$$\begin{aligned} F_{pb1} &= F_{pbx} / \cos\theta \\ F_{pb2} &= F_{pby} / \sin\theta \qquad (4) \\ F_{pbm} &= \min(F_{pb1}, F_{pb2}) \end{aligned}$$

where,

F_{pbx}, F_{pby} : Forces due to passive soil pressures in X and Y directions, respectively

The evaluation results are shown in Tables 3.8-96(4) and 3.8-96(5). The calculated factors of safety are similar to those in Table 3.8-96(3) for the two-dimensional approach using 40% of vertical seismic forces. Therefore, the use of 0.4 vertical seismic component in the two dimensional approach (i.e., N-S & Vertical and then E-W & Vertical) is justified for design evaluation.

As for dead load consideration, SRP 3.8.5 has no requirements for dead load reduction in sliding evaluation. The uncertainties in dead load are implicitly accounted for in the required minimum factor of safety. The 90% reduction specified in Note 1 of DCD Tier 2 Table 3.8-15 and ACI 349 is for design of structural members only and therefore it does not apply to the foundation sliding evaluation. However, the 90% reduction is conservatively considered in the calculations shown in Table 3.8-96(3) and in Tables 3.8-96(4) and 3.8-96(5).

Reference:

1. Bowles, Joseph E. Foundation Analysis and Design. 4th Edition. New York: McGraw-Hill, 1988.

Table 3.8-96(2) Cohesions Based on Maximum Soil Bearing Pressure

Building	RBFB			CB		
Soil Condition	Soft	Medium	Hard	Soft	Medium	Hard
Max. Soil Bearing Stress (MPa)	2.7	7.3	5.4	2.8	2.5	2.4
Cohesion coefficient (MPa)	0.15	0.41	0.30	0.16	0.14	0.13

Table 3.8-96(3) Updated Sliding Stability Evaluation Results

<RB>

Building width X	70.0 m					
Building width Y	49.0 m					
Total Weight	2360 MN					
Buoyancy	652 MN					
Soil Condition	Soft		Medium		Hard	
Vertical Seismic Load	676 MN		1159 MN		1103 MN	
Minimum Vertical Load	1202 MN		1008 MN		1031 MN	
	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir
Fv: Horizontal Seismic Force (MN)	*899*	*787*	*1462*	*1619*	*1485*	*1243*
Fub: Bottom Friction Force (MN)	694	694	582	582	773	773
Fc: Effective Cohesion Force (MN)	514	514	1391	1391	1029	1029
Fpb: Passive Pressure for Basemat (MN)	132	188	213	304	539	769
Fdsf: Passive Soil Pressure on Wall (MN)	0	0	0	0	0	0
Fr: Sliding Resistance (=Fub+Fc+Fpb+Fdsf)	*1340*	*1397*	*2186*	*2277*	*2341*	*2572*
FS (=Fr/Fv)	**1.49**	**1.78**	**1.50**	**1.41**	**1.58**	**2.07**

<CB>

Building width X	30.3 m					
Building width Y	23.8 m					
Total Weight	173 MN					
Buoyancy	101 MN					
Soil Condition	Soft		Medium		Hard	
Vertical Seismic Load	91 MN		83 MN		90 MN	
Minimum Vertical Load	18 MN		22 MN		19 MN	
	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir
Fv: Horizontal Seismic Force (MN)	*124*	*124*	*109*	*118*	*115*	*122*
Fub: Bottom Friction Force (MN)	11	11	12	12	14	14
Fc: Effective Cohesion Force (MN)	112	112	100	100	96	96
Fpb: Passive Pressure for Basemat (MN)	36	46	64	82	173	220
Fdsf: Passive Soil Pressure on Wall (MN)	0	0	0	0	0	0
Fr: Sliding Resistance (=Fub+Fc+Fpb+Fds)	*159*	*169*	*177*	*195*	*283*	*331*
FS (=Fr/Fv)	**1.28**	**1.36**	**1.63**	**1.64**	**2.46**	**2.71**

Table 3.8-96(4) Sliding Evaluation Results for 3-dimensional Inputs: RBFB

Building width X	70.0 m					
Building width Y	49.0 m					
Total Weight	2360 MN					
Buoyancy	652 MN					
Soil Condition	Soft		Medium		Hard	
Vertical Seismic Load	676 MN		1159 MN		1103 MN	
Minimum Vertical Load	1202 MN		1008 MN		1031 MN	
	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir
<3-dimenaional Evaluation> 1.0*NS+0.4*EW+0.4*V						
Factored Horizontal Seismic Force (MN)	*899*	*315*	*1462*	*648*	*1485*	*497*
Fvr: Resultant Seismic Force (MN)	*953*		*1599*		*1566*	
Fub: Bottom Friction Force (MN)	*694*		*582*		*773*	
Fc: Effective Cohesion Force (MN)	*514*		*1391*		*1029*	
Fpb1, Fpb2: Passive Pressure for Basemat (MN)	*142*	*507*	*229*	*819*	*580*	*2072*
Fpbm=min(Fpb1, Fpb2) (MN)	*142*		*229*		*580*	
Fr: Sliding Resistance (=Fub+Fc+Fpbm)	*1350*		*2203*		*2382*	
FS (=Fr/Fv)	**1.42**		**1.38**		**1.52**	
<3-dimenaional Evaluation> 0.4*NS+1.0*EW+0.4*V						
Factored Horizontal Seismic Force (MN)	*360*	*787*	*585*	*1619*	*594*	*1243*
Fvr: Resultant Seismic Force (MN)	*865*		*1721*		*1378*	
Fub: Bottom Friction Force (MN)	*694*		*582*		*773*	
Fc: Effective Cohesion Force (MN)	*514*		*1391*		*1029*	
Fpb1, Fpb2: Passive Pressure for Basemat (MN)	*355*	*203*	*573*	*328*	*1450*	*829*
Fpbm=min(Fpb1, Fpb2) (MN)	*203*		*328*		*829*	
Fr: Sliding Resistance (=Fub+Fc+Fpbm)	*1411*		*2301*		*2631*	
FS (=Fr/Fv)	**1.63**		**1.34**		**1.91**	

Table 3.8-96(5) Sliding Evaluation Results for 3-dimensional Inputs: CB

Building width X	30.3 m					
Building width Y	23.8 m					
Total Weight	173 MN					
Buoyancy	101 MN					
Soil Condition	Soft		Medium		Hard	
Vertical Seismic Load	91 MN		83 MN		90 MN	
Minimum Vertical Load	18 MN		22 MN		19 MN	
	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir
<3-dimenaional Evaluation> 1.0*NS+0.4*EW+0.4*V						
Factored Horizontal Seismic Force (MN)	*124*	*49*	*109*	*47*	*115*	*49*
Fvr: Resultant Seismic Force (MN)	*133*		*118*		*125*	
Fub: Bottom Friction Force (MN)	*11*		*12*		*14*	
Fc: Effective Cohesion Force (MN)	*112*		*100*		*96*	
Fpb1, Fpb2: Passive Pressure for Basemat (MN)	*39*	*123*	*69*	*221*	*187*	*594*
Fpbm=min(Fpb1, Fpb2) (MN)	*39*		*69*		*187*	
Fr: Sliding Resistance (=Fub+Fc+Fpbm)	*162*		*182*		*297*	
FS (=Fr/Fv)	**1.21**		**1.54**		**2.38**	
<3-dimenaional Evaluation> 0.4*NS+1.0*EW+0.4*V						
Factored Horizontal Seismic Force (MN)	*50*	*124*	*43*	*118*	*46*	*122*
Fvr: Resultant Seismic Force (MN)	*133*		*126*		*130*	
Fub: Bottom Friction Force (MN)	*11*		*12*		*14*	
Fc: Effective Cohesion Force (MN)	*112*		*100*		*96*	
Fpb1, Fpb2: Passive Pressure for Basemat (MN)	*97*	*49*	*173*	*88*	*466*	*237*
Fpbm=min(Fpb1, Fpb2) (MN)	*49*		*88*		*237*	
Fr: Sliding Resistance (=Fub+Fc+Fpbm)	*172*		*201*		*348*	
FS (=Fr/Fv)	**1.29**		**1.59**		**2.67**	

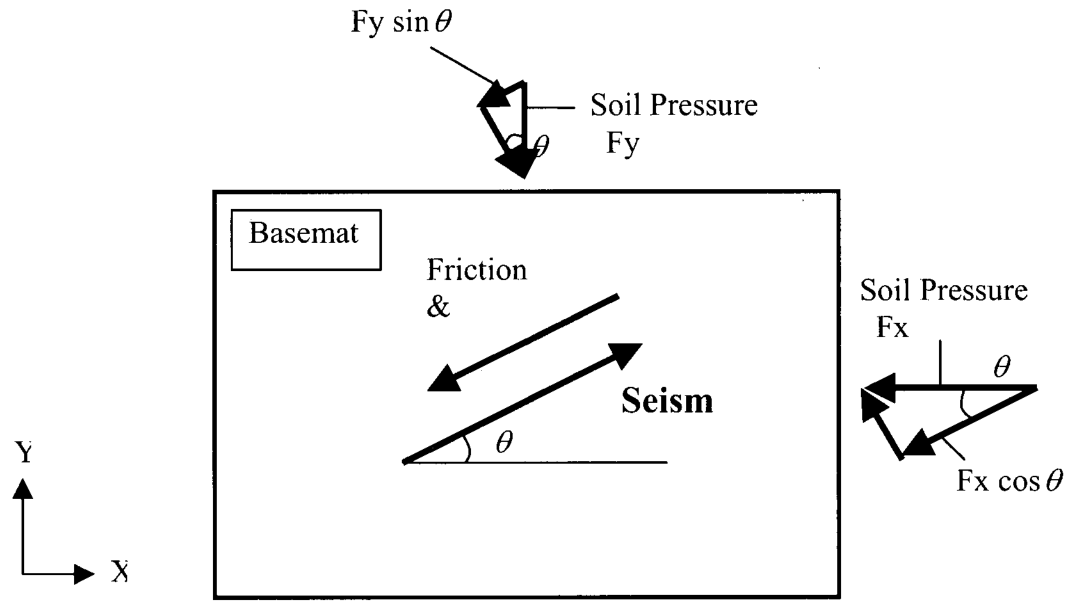


Figure 3.8-96(1) Horizontal Forces in Sliding Evaluation (Basemat Plan)

DCD Impact

No DCD change was made in response to this RAI Supplement.

NRC RAI 3.8-96, Supplement 3

The RAI Supplement 2 response, transmitted in GEH letter dated November 28, 2007, provided information to address nine items related to the stability analyses performed for the ESBWR foundations. The staff requests GEH to address the items discussed below which are still unresolved. The item numbers match the prior RAI Supplement 2 item numbers except for item number 10 which is a follow-up item from RAI 3.8-96, Supplement 1. Note that some of the items discussed below, in the context of sliding stability, are also applicable to overturning stability.

(1) In the equation given for passive soil pressure, why was the water pressure considered in resisting sliding, since there would be an equal and opposite water pressure on the other side of the building? Why wasn't the active soil pressure, on the entire foundation wall and basemat vertical edge, due to static and seismic loads considered on the other side of the building acting in the opposite direction to the passive pressures? Clearly define what surcharge loads (q) were utilized in the equation, because only known permanent surcharge loads (e.g., from other buildings) which would never be removed are appropriate.

(2)

a. GEH states that the shear strength of the soil, i.e., the resistance at the basemat bottom, is composed of friction and cohesion. However, the procedure described by GEH would only apply to a sliding capacity calculation where failure occurs within the soil medium; it would not apply to a sliding capacity calculation at the concrete to soil interface. Therefore, GEH also needs to consider the sliding capacity caused by sliding resistance between the concrete and soil interface (alone). Typically this consists of the bottom friction resistance term given in Tables 3.8-96(3) and 3.8-96(4) of the RAI response which is identified as "Fub: Bottom Friction Force." If any additional sliding resistance due to cohesion between the soil and concrete at the foundation bottom is used, then describe this approach and explain how it compares to other industry analytical methods such as the Navy Design Manual DM7-02 (available from various websites). Such an approach would require having a cohesive soil which would then become a site interface parameter. This will then need to be placed in DCD Tier 1 and Tier 2, and will need to be satisfied by the COL applicant. Note that whatever approach is used for all soil stability calculations, the evaluations must cover all soil types/conditions that the design certification is intended to cover (e.g., soft, medium, and hard soils; cohesive soils and granular (cohesionless) soils; varying soil friction angle; etc.).

b. For the case of sliding frictional resistance capacity between the foundation mat and soil, the staff does not agree that the use of the static coefficient of friction is conservative. The shear force required to initiate sliding between two surfaces is usually greater than the force required to maintain motion, and therefore it is not conservative to use the higher value to resist sliding.

Furthermore, the use of the static frictional resistance at the bottom of the basemat is not consistent with the use of the passive soil resistance at the vertical edge of the basemat. This is because to mobilize the full passive resistance at the vertical edge of the basemat requires some movement of the basemat, in which case, the dynamic sliding friction would be more applicable. Based on the above, GEH is requested to revise their approach to ensure that all of the resisting forces utilized to prevent sliding are developed using a consistent set of assumptions or provide justification for any alternative methods.

(3) No additional information needed.

(4) The equation provided for the calculation of cohesion (c') for use in sliding evaluations does not appear to be appropriate for its intended use. That is because of the following items: (a) It appears that this equation which determines the cohesion value c' is only applicable for cohesive soils, not granular (cohesionless) soils; (b) The use of the cohesion value is applicable for soil shear capacity calculations where failure may occur within the soil medium; it would not be applicable for a sliding capacity calculation at the concrete to soil interface; (c) The relationship between q_u *and cohesion c' and the recommended use of 0.5 to 0.7 of c' for sliding stability evaluations could not be located in Reference 1, which was referred to in the RAI response; (d) The magnitudes of the bearing capacities tabulated in Table 3.8-96(2), which are used to determine c' seem to be unrealistically high. They would require, for the RB/FB medium soil case for example, a soil bearing pressure capacity of 7.3MPa (153ksf) which are extremely large compared to known soil and rock capacities (also identified under RAI 3.8-94). Therefore, GEH is requested to provide the technical basis for application of their approach for all soil types/conditions (e.g., soft, medium, and stiff; cohesive soils and granular (cohesionless) soils; varying soil friction angle; etc.) that the design certification is intended to cover or utilize other accepted analytical methods typically used for sliding evaluations as discussed under item (2) above.*

(5) and (6) Please revise the response to these items based on any revision to Item (4).

(7) The reduction in contact area between the foundation basemat and the soil, due to some overturning uplift from seismic loads, needs to be considered in the calculations, especially since the margins currently shown in the tables will change and may be reduced when the sliding calculations are revised to address the other items in this RAI.

(8)

a. Confirm whether the response given means that the analysis and design of the SSCs in the ESBWR plant including development of the floor response spectra were all based on the enveloped responses for the lumped mass models and the SASSI models. If the analysis and design of the SSCs were based only on the lumped mass models, then did all of the building responses (i.e., member forces, nodal accelerations, nodal displacements, and floor *response spectra) from the lumped mass models bound the responses from the SASSI models?*

b. *From the response to this item, it appears that the shear wave velocity of the backfill material does not have to match the surrounding undisturbed soil. Since the properties of the backfill material will likely be different, GEH is requested to identify the extent of excavation of the soil during the construction of the plant structures and identify what will be the requirements for the soil properties of the backfill material. If these are different than what were assumed in any of the seismic analyses and designs, then GEH is also requested to provide the technical basis for accepting the differences or confirm that the design basis building responses (including floor response spectra) bound the expected values of the backfill soil properties (including reduced shear wave velocities). In the case of the foundation walls, GEH is also requested to explain why the elastically calculated wall pressures from seismic and other loads are still appropriate in view of the soil properties (including reduced shear wave velocity) of the backfill material. Unless the analyses and design cover the entire range of possible backfill soil properties, the assumed soil properties for the backfill materials should be considered a requirement, and therefore, clearly stated in the DCD as a site requirement.*

(9) As noted in the staff's prior assessment of GEH RAI 3.8-96, Supplement 2, response, the traditional method for evaluating the stability (sliding and overturning) of nuclear plant structures in accordance with SRP 3.8 is to perform two separate 2-D evaluations, one for the N-S direction and one for the E-W direction. The minimum vertical downward load (deadweight minus upward buoyancy force minus upward vertical seismic force) is considered separately with the N-S horizontal seismic force and with the E-W horizontal seismic force.

In calculating the total upward vertical seismic force, the total N-S horizontal seismic force, and the total E-W horizontal seismic force at the soil/foundation interface, it is acceptable to use either SRSS or 100-40-40 (as defined in RG 1.92, Rev. 2) to combine the individual RESPONSES from response spectrum analyses for the 3 directions of seismic loading. Thus, the SRSS or the 100-40-40 methods are used only to determine the individual total structural response in a given direction (e.g., total shear force in N-S direction) from the individual collinear responses due to each of the three perpendicular seismic excitations (i.e., N-S shear force due to N-S earthquake, N-S shear force due to E-W earthquake, and N-S shear force due to vertical earthquake). The approach GEH is using does not follow this method, but instead combines non-collinear structural responses (i.e., N-S shear force, E-W shear force, and vertical force) following the 100-40-40 method, which is unacceptable. In lieu of this, the results from a 3-D time history analysis using statistically independent inputs can be used, to search the time history response for the worst case combination of vertical and horizontal seismic responses, which minimize the sliding and overturning factors of safety when combined with deadweight and upward buoyancy force.

GEH's proposed application of the 100-40-40 method in this case is not consistent with the staff's acceptance of the method, which as stated in RG 1.92, Rev. 2, applies to combination of individual response components when RSA is used. On this basis, it is not acceptable to the staff. The two approaches described above are acceptable. If GEH chooses to apply an alternate method, then it will need to submit a comparison to results that would be achieved by either one of the two methods described above.

(10) The crystalline powder which is proposed by GEH for use in the mud mat concrete below the basemat and which is intended to provide waterproofing to prevent water infiltration or ex-filtration still raises some questions. It appears that the concrete mud mat is unreinforced and therefore, cracking of the mud mat is very likely to occur and the crystalline powder may not be effective in preventing water infiltration or ex-filtration. GEH is requested to provide technical information that demonstrates the effectiveness of the crystalline additive in concrete foundations. This information should include: the requirements necessary for proper use of this product, data which demonstrates its effectiveness under similar conditions (e.g., reinforced or unreinforced concrete, effect on concrete compressive strength, minimum thickness required for the concrete section, water pressure/head capacity and permeability versus water pressure/head, etc.), and what performance testing requirements will need to be satisfied during construction. In addition, specific information needs to be provided in the DCD regarding: the compressive strength of the concrete mud mat, if any reinforcement is needed, the acceptable range of thickness for the concrete mud mat, the inclusion of a statement (which was made in the Supplement 1 response) that "The mud mat construction is performed in accordance with the same standards and requirements as the basemat," and inclusion of performance testing requirements that will be needed during construction of the mud mat (e.g., permeability testing, compressive strength testing, etc.). GEH is also requested to explain what waterproofing system is relied upon to prevent infiltration of ground water through the walls below grade.

Revised GEH Response

(1) The water pressure term in the passive pressure equation described in the response to NRC RAI 3.8-96, Supplement 2 was not considered in resisting sliding. The effect of active soil pressure is considered in the revised sliding evaluation (see Item 9 for details) in terms of a net lateral resistance pressure (i.e., the difference between passive and active pressures) that is required to achieve minimum 1.1 factor of safety against sliding. In this revised sliding evaluation, the permanent surcharge loads from the Turbine Building are also included as lateral soil force applied to the RB/FB.

(2)

a. See Item (9) on the revised sliding evaluation approach in which the cohesion resistance is ignored

b. See item (9) on the revised sliding evaluation approach in which all of the resisting forces utilized to prevent sliding and associated site interface parameters are defined.

(3) In the NRC Audit in June 2008, the staff requested the following additional information.

For the sliding resistance between the basemat and mudmat, GEH needs to provide the technical basis for the coefficient of friction of 0.7. Currently ACI 349 Section 11.7.4.3 which states that mu is 0.6 concrete placed on concrete with surface not intentionally roughened and 1.0 if the surface is intentionally roughened as specified in 11.7.9 (roughened to ¼ inch).

The weak link at the sliding interface of concrete to soil is the soil, since the concrete surface in contact with soil is rough. As a result, the 0.7 coefficient of friction is controlled by the soil shear strength as a function of internal friction angle, tan (ϕ), where ϕ is equal to 35 degrees. Since this friction angle results in a friction coefficient larger than 0.6, which is the value for concrete placed against hardened concrete not intentionally roughened in accordance with ACI 349 Section 11.7.4.3, roughening the mudmat top surface is required to ensure that the interface between the basemat and mudmat is not the controlling sliding surface. The following statement, “The top surface of the mudmat is intentionally roughened in accordance with ACI 349-01 Section 11.7.9 requirement.” will be added to DCD Tier 2 Subsection 3.8.6.5.

(4) The equation for the calculation of cohesion (c’) is no longer used in the revised sliding evaluation in Item (9).

(5) and (6) See Item (4).

(7) The reduction in contact area between the foundation basemat and the soil, due to some overturning uplift from seismic loads, is considered in a separate calculation of bearing pressures in the response to RAI 3.8-94 S03, transmitted to the NRC on December 9, 2008 via MFN 06-407, Supplement 10.

(8)

a. The building responses are all based on the enveloped responses for the lumped mass models and the SASSI models.

b. The effects of backfill adjacent to building walls on structural response can be addressed in two aspects. One deals with the global SSI effect and other with the local wall pressures. For the global SSI effect, the design forces are controlled by non-embedded cases using lumped mass model as shown in DCD Tier 2 Subsection 3A.8.7. This has been further confirmed by additional SASSI analyses for uniform sites taking into account embedment as discussed in RAI 3.8-94 S03. The effect of embedment on the design floor response spectra, as discussed in RAI 3.8-94 S03 is only limited to high frequency range at few locations in the CB and FPE. Inclusion of high frequency response in the design response spectra is a conservative design requirement without consideration of the beneficial effects of seismic wave incoherence. Therefore, it can be concluded that for the purpose of the global SSI response, no additional site interface requirements for the property of backfill material are needed in the DCD. For the local effect on wall lateral pressures, the main parameters are the density, Poisson's ratio and peak ground acceleration in accordance with the ASCE 4-98 Section 3.5.3.2 Elastic Solution method. To ensure the wall design seismic lateral pressures induced from backfill are not exceeded, a COL item will be added in DCD Tier 2 Table 2.0-1 to limit the product of peak ground acceleration (α) of the site-specific Foundation Input Response Spectra (FIRS) in g's, Poisson's ratio (ν) and density (γ) as follows:

$\alpha\,(0.95\nu + 0.65)\,\gamma$:   1220 kg/m^3 (76 lbf/ft^3) maximum

Additional site interface parameters for backfill related to sliding are defined in Item (9) below.

(9) This part of the RAI response presents the revised sliding evaluation. Time-consistent phasing between the horizontal base shear and vertical base force is considered to compute the sliding factor of safety as a function of time when combined with deadweight and upward buoyancy force. In this evaluation the base shears and base vertical forces calculated by SASSI analyses with embedment included are used. See RAI 3.8-94 S03 for details of additional SASSI analyses for uniform sites.

1. Soil Properties

The following soil properties are assumed in the sliding evaluation. They will be stated in the DCD Table 2.0-1 as site interface requirements.

- Angle of internal friction
  ϕ = 35 degree minimum for all sites
- Backfill on sides of Seismic Category I structures (not applicable if the fill material is concrete)

Product of at-rest soil pressure coefficient (k_o) and density (γ)

$k_0\gamma$:   750 kg/m^3 (47 lbf/ft^3) minimum

Product of the difference of passive (k_p) and active pressure (k_a) coefficients and density (γ)

$(k_p-k_a)\gamma$: 1100 kg/m^3 (69 lbf/ft^3) minimum

- Backfill underneath FWSC against shear keys (not applicable if the fill material is concrete)

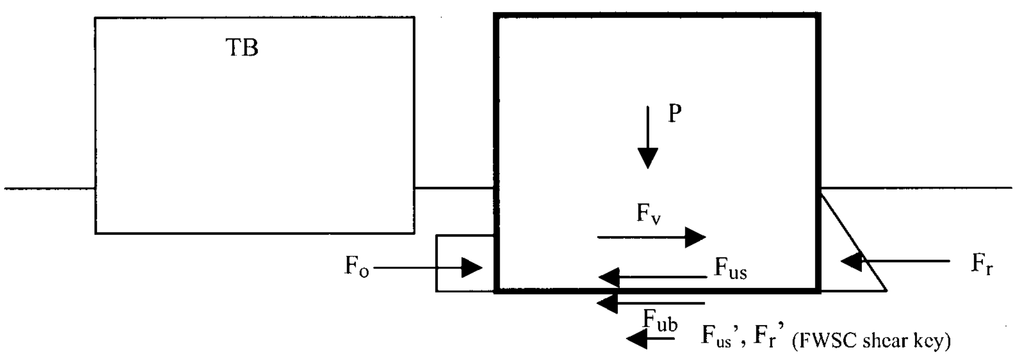
At-rest pressure coefficient (k_o')

k_0' : 0.36 minimum

Difference of passive (k_p') and active pressure (k_a') coefficients

$(k_p'-k_a')$: 2.5 minimum

2. Sliding Evaluation Method



FS (factor of safety) is evaluated by taking the minimum values of the *FS(t)* time history calculated per the following equation.

$$FS(t) = \frac{F_{ub}(t) + F_{us} + F_r + F_{us}' + F_r'}{F_v(t) + F_o} \quad \text{.......(1)}$$

where,

$F_v(t)$: Base shear time history at bottom of basemat.

F_o: Lateral soil force on RB due to TB surcharge load.

$F_{ub}(t)$: Friction resistance force provided by basemat bottom.

For "Dry sites" where ground water is below the foundation:

$F_{ub}(t) \quad = P\ tan\phi = (0.9D\text{-}V_z(t))\ tan\phi$

For "Wet sites" where ground water is above the foundation:

$F_{ub}(t) \quad = P\ tan\phi = (0.9D\text{-}B)\ tan\phi$ (undrained shear strength)

where D: Dead weight

$V_z(t)$: Vertical seismic force time history

B: Buoyancy

The vertical seismic force is not considered in the building stability calculations under the undrained seismic event. The peaks in seismographic strong motion time histories last only for hundredths of seconds which is at least an order of magnitude less than the time it takes to adjust pore pressures. The delay in adjustment of pore pressures results in that there is not enough time for the pore fluid to accommodate the changes in pore water pressure and the effective normal stress does not change, and hence, the shear strength does not change either. Therefore, the undrained shear strength is not affected by the vertical seismic loading.

F_{us}: Skin Friction resistance force provided by basemat side parallel to the direction of motion.

$F_{us} = P_0\ tan\phi$ ....(2)

where,

$P_0 = k_o \gamma LH^2/2$: At-rest soil force on the basemat side neglecting surcharge term and water pressure term

where, L: Length of basemat parallel to the direction of motion

H: Embedment depth

F_r: Lateral resistance pressure along the wall and basemat normal to the direction of motion.

Additional sliding resistance is provided by the side soil and it is defined to be the difference of the passive and active pressures. The net resistance is determined to achieve the required 1.1 FS, while not exceeding the at-rest soil pressure considered in the wall design.

$F_r = (k_p\text{-}k_a)\gamma LH^2/2$ ....(3)

where, L: Length of building normal to the direction of motion

H: Embedment depth

F_{us}': Skin Friction resistance force provided by FWSC shear-key side parallel to the direction of motion.

$F_{us}' = P_0' \tan\phi$ ....(4)

where,

$P_0' = k_o' qL'H'$: At-rest soil force on the FWSC shear-key side

where, q: FWSC surcharge load

L': Length of shear-key parallel to the direction of motion

H': Shear-key depth

F_r': Lateral resistance pressure along FWSC shear-key normal to the direction of motion. The net resistance is determined to achieve the required 1.1 FS.

$F_r' = (k_p' - k_a')\, qL'H'$: ....(5)

where, q: FWSC surcharge load

L': Length of shear-key normal to the direction of motion

H': Shear-key depth

3. Summary of Calculated FS

Summary

(1) Dry condition

	L-1		L-2		L-3		L-4		SOFT		MEDIUM		HARD	
	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir
RB/FB	1.86	3.50	-	-	2.30	3.42	-	-	2.43	3.04	1.68	2.27	1.98	2.54
CB	2.10	1.97	-	-	2.11	2.04	-	-	2.17	2.09	1.61	1.63	1.58	1.84
FWSC (H=3.0m)	1.27	1.33	1.10	1.34	1.28	1.49	1.12	1.28	1.28	1.48	1.27	1.33	1.12	1.18

(2) Undrained condition

	L-1		L-2		L-3		L-4		SOFT		MEDIUM		HARD	
	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir
RB/FB	1.66	2.87	-	-	1.86	2.89	-	-	1.92	2.51	1.53	2.05	1.66	2.04
CB	1.42	1.33	-	-	1.41	1.39	-	-	1.44	1.40	1.14	1.15	1.10	1.11
FWSC (H=3.0m)	1.45	1.46	1.33	1.57	1.53	1.67	1.33	1.54	1.50	1.62	1.55	1.63	1.44	1.62

Minimum FS

	Minimum
RB/FB	1.53
CB	1.10
FWSC	1.10

Cases L-2 and L-4 are not considered for RB/FB and CB. To be consistent with this limitation, a new site interface parameter for maximum ratio of shear wave velocity in adjacent layers will be added in DCD Tier 2 Table 2.0-1 to ensure that the site layering does not have large contrast in shear wave velocities as generic layer sites L-2 and L-4 (see DCD Tier 2 Table 3A-3 for descriptions of layered sites) as follows:

Bottom 20 m (66 ft) layer to top 20 m (66 ft) layer: 2.5

Bottom 40 m (131 ft) layer to top 20 m (66 ft) layer: 2.5

Adjacent layers are the two layers with a total depth of 40 m (131 ft) or 60 m (197 ft) below grade. The first layer, termed top layer, covers the top 20 m (66 ft). The second layer, termed bottom layer, covers the next 20 m (66 ft) or 40 m (131 ft). The ratio is the average velocity of the bottom layer divided by the average velocity of the top layer. Either the lower bound seismic strain (i.e., strain compatible) profile or the best estimate low strain profile can be used since only the velocity ratio is of interest. This velocity ratio condition does not apply to the FWSC nor to the RB/FB and CB if founded on rock-like material having a shear wave velocity of 1067 m/sec (3500 ft/sec) or higher.

(10)

The integral crystalline material waterproofs and protects concrete in-depth and is applied as an admixture to the mud mat concrete mix at the time of batching. The crystalline waterproofing material can self-heal cracks up to 0.4 mm.

As an added waterproofing measure for any mud mat cracks exceeding 0.4 mm during basemat construction, once the mud mat has cured and just before pouring the basemat, the crystalline waterproofing material will be applied at the top surface of the mud mat. Once the basemat is poured, this added crystalline waterproofing material will penetrate into the mud mat to self-heal concrete cracks. In addition, any mud mat cracks will also be filled by the basemat cement paste.

Calculated maximum crack widths for the mud mat during normal conditions and for the basemat during construction and normal conditions are contained in Table 3.8-96(6). The basemat is designed to limit the concrete crack width during construction and normal conditions to no more than 0.4 mm.

Technical information that demonstrates the effectiveness of crystalline waterproofing material for concrete, including the requirements necessary for proper use of the product, data which demonstrates its effectiveness, and necessary performance testing requirements that need to be satisfied during construction, are attached as Attachment 3.8-96, Supplement 3(X), Attachment 3.8-96, Supplement 3(Y) and Attachment 3.8-96, Supplement 3(Z).

The mud mat is designed as structural plain concrete in accordance with ACI 318-05. The specified compressive strength of concrete at 28 days, or earlier, is 2500 psi for the mud mat. The thickness of the mud mat is no less than 8 inches. The performance testing requirements for the mud mat are those delineated in ACI 318-05. The mud mat construction is performed in accordance with the same standards and requirements as the basemat. These mud mat details will be added as DCD Tier 2 Subsection 3.8.6.5 in Revision 6.

As stated in the response to NRC RAI 3.8-89, which was transmitted to the NRC via MFN 06-407 on November 8, 2006, a membrane waterproofing system is applied to the exterior walls and is relied upon to prevent infiltration of ground water through the exterior walls below grade.

Table 3.8-96(6) Calculated Maximum Crack Widths for Basemat and Mud-mat

	*During Construction *1*	*During Normal Condition*
Basemat	*0.13 mm*	*0.12 mm*
Mud-mat	---	*0.17 mm*

Note *1: Crack width at the basemat bottom of the first concrete layers during the second concrete pouring were calculated, based on the results of analyses performed for RAI 3.8-93 response.

DCD Impact

DCD Tier 1 Table 5.1-1 will be revised in Revision 6 as noted in the attached markup.

DCD Tier 2 Subsection 3.8.6.5 will be added, Tables 2.0-1, Subsections 3G.1.5.5, Table 3G.1-57, Subsections 3G.2.5.5, Table 3G.2-26, Subsections 3G.4.5.5, and Table 3G.4-22 will be revised, and Figures 3G.1-65, 3G.2-17, and 3G.4-11 will be deleted as noted in the attached markup. These changes will be made in Revision 6 of DCD Tier 2.

NRC RAI 3.8-96, Supplement 4

Based on the review of GEH RAI 3.8-96 S03 response, presented in GEH letter dated February 20, 2009, GEH is requested to address the items described below.

A) *In response to Item 3 on Page 21 of 27, the following statement is made. "The weak link at the sliding interface of concrete to soil is the soil, since the concrete surface in contact with soil is rough. As a result, the 0.7 coefficient of friction is controlled by the soil shear strength as a function of internal friction angle, tan (□), where □ is equal to 35 degrees. Since this friction angle results in a friction coefficient larger than 0.6, which is the value for concrete placed against hardened concrete not intentionally roughened in accordance with ACI 349 Section 11.7.4.3, roughening the mudmat top surface is required to ensure that the interface between the basemat and mudmat is not the controlling sliding surface. The following statement, "The top surface of the mudmat is intentionally roughened in accordance with ACI 349-01 Section 11.7.9 requirement." will be added to DCD Tier 2 Subsection 3.8.6.5."*

   This response however, appears to neglect potential sliding between the bottom of the mud mat and the soil surface, and implies that sliding will take place in the soil below the mud mat. GEH is requested to provide the technical basis for the statement that "the concrete surface in contact with the soil is rough", and as a result, the failure surface can only occur within the soil below the mud mat (e.g., providing appropriate references and/or test data). Alternatively, testing by the COL applicant may be required to demonstrate this assumption.

B) *In Item (8) (page 21 of 27), GEH indicates that the design forces on the walls of the NI are based on the envelope of SASSI runs for non-embedded cases using uniform half-space representations of a site as well the results of two layered soil cases using the embedded condition of the NI. Provide the following information for the embedded soil cases: (1) explain whether the input motions were defined at the basemat elevation, (2) if so, explain how the motions were converted to the appropriate input motions in SASSI problem, and (3) explain why the results of two layered cases can be considered as bounding for generic design. Also see requested information in new RAIs 3.7-69 and 71, that relate to this issue.*

   In the same section, GEH also provides the following recommendation: "To ensure the wall design seismic lateral pressures induced from backfill are not exceeded, a COL item will be added in DCD Tier 2 Table 2.0-1 to limit the product of peak ground acceleration (α) of the site-specific Foundation Input Response Spectra (FIRS) in g's, Poisson's ratio (ν) and density (γ) as follows: α (0.95ν + 0.65) γ: 1220 kg/m3 (76 lbf/ft3) maximum." Provide an explanation and the basis for this recommendation.

C) *In Item (9) (pages 22 through 26 of 27), a description of the revised sliding evaluation is presented. This new calculation considers the static coefficient of friction beneath the basemat and on the side walls, passive soil pressures, and at rest soil pressures. As indicated in the prior revision to this RAI, the use of these terms should be based on a consistent set of expected deformations. For example, to develop the full passive pressure capability of the soil implies that sufficient* foundation deformation occurs. This may not be consistent with the use of the full static coefficient of friction. Therefore, provide detailed information which demonstrates that the individual forces used in the stability calculations are calculated in a consistent manner for the assumed foundation displacements.

D) In Item (9), (page 24 of 27), the lateral resistance pressure (Fr) provided by the foundation/walls perpendicular to the direction of motion is defined to be the difference of the passive and active pressures. The paragraph also states that "The net resistance is determined to achieve the required 1.1 FS, while not exceeding the at-rest soil pressure considered in the wall design." For the FWSC, another term Fr' is defined as: "Lateral resistance pressure along the FWSC shear-key normal to the direction of motion. The net resistance is determined to achieve the required 1.1 FS." In Section 3 – Summary of Calculated FS, presented on page 25 of 27 of the RAI response, the minimum FS for the RB/FB is equal to 1.53, and for the CB and FWSC the FS is 1.1. GEH is requested to address the related items listed below.

(a) For the RB/FB, if Fr is calculated such that the FS is equal to 1.1, explain why the Summary of Calculated FS in the RAI response states that FS is equal to 1.53 and not 1.1.

(b) Explain why Fr "is determined to achieve the required 1.1 FS, while not exceeding the at-rest soil pressure considered in the wall design." According to the DCD, the foundation walls are designed for the worst soil pressures resulting from either SASSI 2000 analysis or ASCE 4-98 methodology, not the at-rest soil pressure.

(c) For Fr' (used for the FWSC), there is no limitation on exceeding the at-rest soil pressure considered in the wall design, as there is for the other structures. Confirm that this was intended to be the case. If so, then were the shear keys designed for this potentially higher passive pressure load?

(d) In view of the confusion, for each of the three structures (RB/FB, CB, and FWSC), provide a description of the approach used to calculate each of the resisting forces, their calculated magnitudes (for the governing FS), and compare the total calculated pressures for these resisting forces to what were used in the actual design. This comparison should clearly demonstrate that the foundation walls were designed to the higher of the SASSI 2000 analysis, ASCE 4-98 methodology, and sliding stability required passive pressures.

E) In Item (9) (page 24 of 27), the lateral resistance provided by the foundation/walls <u>parallel to the direction of motion</u> (i.e., vertical edges of the side foundation/walls) is given as $F_{us} = P_o \tan(\phi)$, where □ is the soil internal friction angle. Since waterproofing membrane will be used on the vertical edges of the foundation and walls, explain how will it be demonstrated that the coefficient of friction between soil and the membrane is greater than 0.7 (based on $\tan(\phi)$, where ϕ = 35 degrees for the soil).

F) In the description of the sliding evaluation method presented on page 24 of 27, the effective friction angle for wet sites is indicated to be determined from undrained *shear strength data. If, as indicated in the RAI responses provided by GEH, effective pore pressures under seismic conditions are deemed to remain unchanged during short seismic response times, explain why the effective friction angle is not defined as potentially zero, particularly for silty foundation soils.*

G) *In Item (10) (page 26 of 27), GEH indicates that "The basemat is designed to limit the concrete crack width during construction and normal conditions to no more than 0.4mm." Item (10) also states that "The mud mat is designed as structural plain concrete in accordance with ACI 318-05." Since the concrete is identified as plain concrete, it is not clear whether any reinforcement is utilized in the mud mat. Explain whether the design of the mud mat includes sufficient reinforcement: to limit cracks to no more than 0.4mm and to address temperature and shrinkage effects in accordance with ACI code requirements. Identify where the reinforcement requirements for the mud mat are defined in the DCD.*

H) *In Item (10) (page 26 of 27), GEH indicates that a membrane waterproofing system is applied to the exterior walls and is relied upon to prevent infiltration of ground water through the exterior walls below grade. This does not address the RAI question which asked what waterproofing system is relied upon. GEH should provide information such as the type of waterproofing material, thickness, and whether the provisions of an industry standard such as ACI 515.1R-79 (revised 1985) will be used.*

I) *GEH is requested to revise other applicable sections of the DCD (Section 3.8 and related appendices) that are affected by the revised calculation for sliding stability. As an example, DCD Tier 2, Section 3.8.5.5 – Structural Acceptance Criteria does not reflect the current approach being used.*

GEH Response

A) The assumed 0.7 coefficient of friction can be achieved as long as the angle of internal friction, which is a site interface requirement, is no less than 35 degrees. In order to ensure that the failure surface can only occur within the soil below the mud mat and to justify the use of a 0.7 coefficient of friction, troughs are provided on the ground surface before the mud mat is poured. The size of the troughs is approximately 150 mm (6 in) wide and 100 mm (4 in) deep. They are arranged in a grid pattern with no larger than a 2.5 m (8.2 ft) spacing distributed over the footprint of the mud mat. The trough size and spacing are determined such that the mud mat concrete shear stress due to the friction forces is less than the ACI 349-01 allowable concrete shear stress. The trough requirements will be added to DCD Tier 2 Subsection 3.8.6.5 in Revision 6.

B) The following information is for the embedded soil cases:

(a) The input motions for the embedded soil cases are defined as outcrop motion at the basemat bottom elevation.

(b) These foundation input motions are converted to the surface motions by a SHAKE analysis in which the entire column was used. These surface motions are then used as input motion in SASSI2000.

(c) The two layered site soil Cases L-2 and L-4 are no longer excluded in the soil bearing and sliding evaluations. Please see GEH's response to NRC RAI 3.8-94 S04 (MFN 09-388, dated 6/12/09).

The seismic lateral pressure limit, $\alpha\ (0.95\nu + 0.65)\ \gamma$, is derived from the resultant force F_r equation in ASCE 4-98, Figure 3.5-2, as follows:

$F_r = \alpha C_v \gamma H^2$ (from Figure 3.5-2 of ASCE 4-98)

where,

α: horizontal earthquake acceleration (g)

γ: soil unit weight

H: embedment height

C_v: coefficient as a function of Poisson's ratio, ν. A numerical analysis of this equation shows that C_v, the coefficient as a function of Poisson's ratio, can be approximated by a straight line, $0.95\nu + 0.65$, as shown in Figure 3.8-96(4).

C) The magnitude of foundation deformation is evaluated for wall rotation as a ratio of the horizontal displacement at grade relative to base to the height of the embedded wall. Among all SASSI results, the maximum rotation of the embedded RB/FB and the CB are 0.0008 (0.08%) and 0.0002 (0.02%), respectively, which are much smaller than the wall movement required for the development of passive pressures in accordance with Figure 1 in Chapter 3 of the Navy Design Manual 7.02 (NRC RAI 3.8-96 S04, Reference 1). Therefore, the foundation can be treated as being in a non-displaced state using the static coefficient of friction. The individual forces used in the revised stability calculations are calculated in a consistent manner for the non-slide condition. Shear keys are provided as needed to ensure a non-slide condition. Details are presented in the updated sliding evaluation at the end of this supplemental response.

D)

(a) The 1.1 minimum factor of safety (FS) is the most critical for the Seismic Category I structures. In the previous evaluation, the CB is most critical and the RB/FB has a larger FS. As explained in Item C) above, the sliding evaluation will be updated and the FS values will also be revised.

(b) The foundation walls are designed for the combined loads of the at-rest soil pressures and the seismic lateral pressures resulting from the SASSI analysis and ASCE 4-98 elastic solution. In the updated evaluation presented below, F_r is set to be the wall design pressure of at-rest plus seismic.

(c) There is no F_r' limitation for the FWSC because the FWSC has no embedded walls. The shear keys for the FWSC are attached to the bottom of basemat and are designed to the differential pressure between soil passive pressures and active pressure, k_p-k_a.

(d) The sliding evaluation approach used and results obtained are described for each structure at the end of this supplemental response.

E) The skin friction, F_{us}, is considered for the basemat only and not for the walls. The vertical edges of the basemat do not use a waterproofing membrane and instead are sprayed with the crystalline waterproofing material to ensure that the 0.7 coefficient of friction is achieved.

F) The vertical seismic responses will be included in all cases. The revised sliding evaluation and results are in the "Detailed Evaluation" below.

G) As stated in Part (10) of GEH's response to NRC RAI 3.8-96 S03 (MFN 06-407 S14, dated 2/20/09), the mud mat is designed as Plain Concrete. The mud mat contains no reinforcement. It is used to provide a level surface for construction. As required by ACI 318-05 Chapter 22, contraction joints will be used to limit the spread of cracking due to creep, shrinkage, and temperature effects. The crystalline waterproofing material will be applied to the top surface of the mud mat as an added waterproofing measure for any mud mat cracks exceeding 0.4 mm during basemat construction. Once the basemat is poured, this added crystalline waterproofing material will penetrate into the mud mat to self-heal concrete cracks. In addition, any mud mat cracks will be filled by the basemat cement paste.

H) The type of the waterproofing system applied to the exterior walls is sheet-applied barrier materials described in Section 4.2.1.4 of ACI 515.1R-79 (revised 1985) (e.g. non-vulcanized butyl rubber sheet). The thickness of the waterproofing sheet is 2.0 mm. Two layers of sheets are applied to the exterior walls below grade.

I) The revised sliding evaluation and results are in the "Detailed Evaluation" below. DCD Tier 2 Subsections 3.8.5.5, 3.8.6.5 and 3G.1.5.5, Tables 2.0-1, 3G.1-57 and 3G.2-26 and Figures 3G.1-1, 3G.1-6, 3G.1-7 and 3G.4-1 will be revised in Revision 6 accordingly.

Reference:

1. Naval Facilities Engineering Command, "Foundations & Earth Structures," Navy Design Manual 7.02, September 1986.

Detailed Evaluation

1. Soil Properties

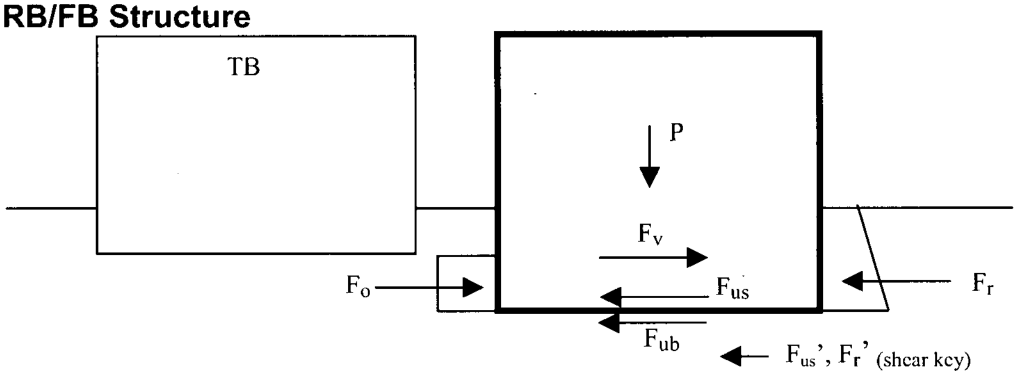
The following soil properties are assumed in the sliding evaluation. They are site parameter requirements for backfill on the sides and underneath of Seismic Category I structures:

- Angle of internal friction
  ϕ = 35 degree minimum
- Soil density
  γ = 1900 kg/m^3 (119 lbf/ft^3) minimum
- At-rest pressure coefficient
  k_o = 0.36 minimum
- Product of at-rest soil pressure coefficient and density
  $k_o\gamma$ = 750 kg/m^3 (47 lbf/ft^3) minimum

2. Sliding Evaluation

Time-consistent phasing between the horizontal base shear and vertical base force is considered to compute the sliding factor of safety (FS(t)) as a function of time when combined with deadweight and upward buoyancy force.

(a) RB/FB Structure



The FS is evaluated by taking the minimum values of the FS(t) time history calculated per the following equation:

$$FS(t) = \frac{F_{ub}(t) + F_{us} + F_r + F_{us}' + F_r'}{F_v(t) + F_o} \quad \text{.......(1)}$$

where,

$F_v(t)$: Base shear time history at bottom of basemat.
F_o: Lateral soil force on RB due to TB surcharge load.
$F_{ub}(t)$: Friction resistance force provided by basemat bottom.

$$F_{ub}(t) = P\ tan\phi = (0.9D - B - V_z(t))\ tan\phi \quad \text{.......(2)}$$

where D: Dead weight

$V_z(t)$: Vertical seismic force time history
B: Buoyancy

F_{us}: Skin friction resistance force provided by basemat side parallel to the direction of motion.

$F_{us} = P_o \tan\phi$ ....(3)

where,

$P_o = k_o \gamma L(H_2^2 - H_1^2)/2$: At-rest soil force on the basemat side neglecting surcharge term and water pressure term

L: Skin friction length of both sides of basemat parallel to the direction of motion

H_1, H_2: Embedment depths at the top and bottom of basemat

F_r: Lateral resistance pressure along the wall and basemat opposite to the direction of motion. It is equal to the wall design lateral pressure, which consists of at-rest static earth pressures and dynamic earth pressures calculated from the SASSI analysis and the ASCE 4-98 elastic solution.

F_{us}': Skin friction resistance force provided by shear key side parallel to the direction of motion.

$F_{us}' = P_o' \tan\phi$ ....(4)

where,

$P_o' = k_o \gamma L'(H_3^2 - H_2^2)/2 + k_o q L'(H_3 - H_2)$: At-rest soil force on the shear key side

q: Surcharge load of RB/FB

L': Skin friction length of both sides of shear key parallel to the direction of motion

H_2, H_3: Embedment depths at the top and bottom of shear key

F_r': Lateral resistance pressure along shear key opposite to the direction of motion.

$F_r' = (k_p - k_a)\gamma L'(H_3^2 - H_2^2)/2 + (k_p - k_a) q L'(H_3 - H_2)$ ....(5)

where,

$k_p = (1+\sin\phi)/(1-\sin\phi)$: Rankine's passive pressure coefficient

$k_a = (1-\sin\phi)/(1+\sin\phi)$: Rankine's active pressure coefficient

q: Surcharge load of RB/FB

L': Length of shear key opposite to the direction of motion

H_2, H_3: Embedment depths at the top and bottom of shear key

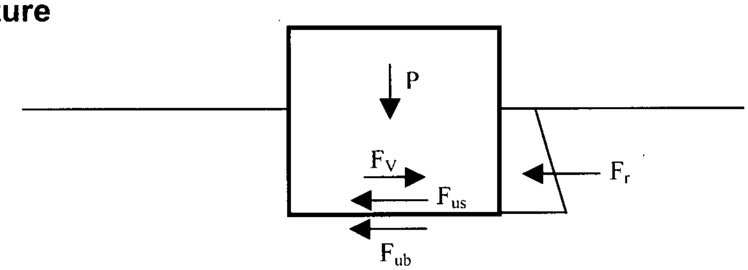
The following are calculation results of individual forces for the RB/FB at the RL-2 site in the NS direction, which is the governing FS case:

$F_v(t)$	=	1,106 MN	(t = 7.175 sec)
F_o	=	128 MN	
$F_{ub}(t)$	=	359 MN	(t = 7.175 sec)
F_{us}	=	52 MN	
F_r	=	497MN	
F_{us}'	=	88 MN	
F_r'	=	391 MN	

$FS = 1.12$

The shear key configuration is shown in Figure 3.8-96(2). The reinforcement in the shear key is determined to resist full capacity of the passive pressure less the active pressure.

(b) CB Structure



The *FS* is evaluated by taking the minimum values of the *FS(t)* time history calculated per the following equation:

$$FS(t) = \frac{F_{ub}(t) + F_{us} + F_r}{F_v(t)} \quad \text{.....(6)}$$

where,

$F_v(t)$: Base shear time history at bottom of basemat.

$F_{ub}(t)$: Friction resistance force provided by basemat bottom.

$F_{ub}(t) = P\ tan\phi = (0.9D - B - V_z(t))\ tan\phi$ .....(7)

where D: Dead weight

$V_z(t)$: Vertical seismic force time history

B: Buoyancy

F_{us}: Skin friction resistance force provided by basemat side parallel to the direction of motion.

$F_{us} = P_o\ tan\phi$ .....(8)

where,

$P_o = k_o \gamma L(H_2^2 - H_1^2)/2$:

At-rest soil force on the basemat side neglecting surcharge term and water pressure term

L: Skin friction length of both sides of basemat parallel to the direction of motion

H_1, H_2: Embedment depths at the top and bottom of basemat

F_r: Lateral resistance pressure along the wall and basemat opposite to the direction of motion. It is equal to the wall design lateral pressure, which consists of at-rest static earth pressures and dynamic earth pressures calculated from the SASSI analysis and the ASCE 4-98 elastic solution.

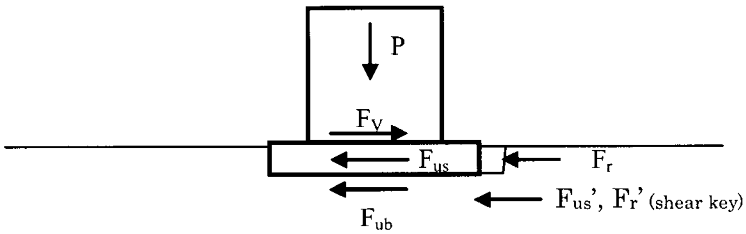
The following are calculation results of individual forces for the CB at the CL-2 site in the NS direction, which is the governing FS case:

$F_v(t)$ = 128 MN (t = 7.375 sec)

$F_{ub}(t)$ = 26 MN (t = 7.375 sec)

F_{us} = 13 MN
F_r = 132 MN
FS = 1.33

(c) FWSC Structure



The FS is evaluated by taking the minimum values of the FS(t) time history calculated per the following equation:

$$FS(t) = \frac{F_{ub}(t) + F_{us} + F_r + F_{us}' + F_r'}{F_v(t)} \quad (9)$$

where,

$F_v(t)$: Base shear time history at bottom of basemat.

$F_{ub}(t)$: Friction resistance force provided by basemat bottom.

$F_{ub}(t) = P \tan\phi = (0.9D - B - V_z(t)) \tan\phi$ ....(10)

where D: Dead weight

$V_z(t)$: Vertical seismic force time history

B: Buoyancy

F_{us}: Skin friction resistance force provided by basemat side parallel to the direction of motion.

$F_{us} = P_o \tan\phi$ ....(11)

where,

$P_o = k_o \gamma L H_1^2 /2$:

At-rest soil force on the basemat side neglecting surcharge term and water pressure term

L: Skin friction length of both sides of basemat parallel to the direction of motion

H_1: Embedment depth of basemat

F_r: Lateral resistance pressure along the wall and basemat opposite to the direction of motion. It is equal to the wall design lateral pressure, which consists of at-rest static earth pressures and dynamic earth pressures calculated from the SASSI analysis and the ASCE 4-98 elastic solution.

F_{us}': Skin friction resistance force provided by shear key side parallel to the direction of motion.

$F_{us}' = P_o' \tan\phi$ ....(13)

where,

$P_o' = k_o \gamma L'(H_2^2 - H_1^2) /2 + k_o q L'(H_2 - H_1)$:

At-rest soil force on the shear key side

q: Surcharge load of FWSC

L': Skin friction length of both sides of shear key parallel to the direction of motion

H_1, H_2: Embedment depths at the top and bottom of shear key

F_r': Lateral resistance pressure along shear key opposite to the direction of motion.

$$F_r' = (k_p-k_a)\,\gamma L'(H_2^2-H_1^2)/2 + (k_p-k_a)qL'(H_2-H_1) \quad (14)$$

where,

$k_p = (1+\sin\phi)/(1-\sin\phi)$: Rankine's passive pressure coefficient

$k_a = (1-\sin\phi)/(1+\sin\phi)$: Rankine's active pressure coefficient

q: Surcharge load of FWSC

L': Length of shear key opposite to the direction of motion

H_1, H_2: Embedment depths at the top and bottom of shear key

The following are calculation results of individual forces for the FWSC at the FL-2 site in the NS direction, which is the governing FS case:

$F_v(t)$ = 104 MN (t = 7.165 sec)
$F_{ub}(t)$ = 41 MN (t = 7.165 sec)
F_{us} = 1 MN
F_r = 4 MN
F_{us}' = 11 MN
F_r' = 57 MN
FS = 1.10

The shear key configuration is shown in Figure 3.8-96(3). The reinforcement in the shear key is determined to resist full capacity of the passive pressure less the active pressure.

3. Summary of Calculated FS

The calculated FS for the RB/FB, CB and FWSC for all site cases are summarized in Table 3.8-96(7).

Table 3.8-96(7) Summary of Factor of Safety for Sliding

	L-1		L-2		L-3		L-4		SOFT		MEDIUM		HARD		Minimum FS
	NS dir.	EW dir.	NS dir.	EW dir.	NS dir.	EW dir.	NS dir.	EW dir.	NS dir.	EW dir.	NS dir.	EW dir.	NS dir.	EW dir.	
RB/FB	2.46	5.24	1.12	1.45	2.95	5.17	1.19	1.49	3.16	4.55	2.23	3.50	2.61	3.90	1.12
CB	2.61	2.84	1.33	1.77	2.62	2.95	1.34	1.76	2.68	3.01	2.02	2.39	1.98	2.57	1.33
FWSC	1.28	1.45	1.10	1.48	1.29	1.65	1.12	1.44	1.29	1.63	1.28	1.49	1.12	1.32	1.10

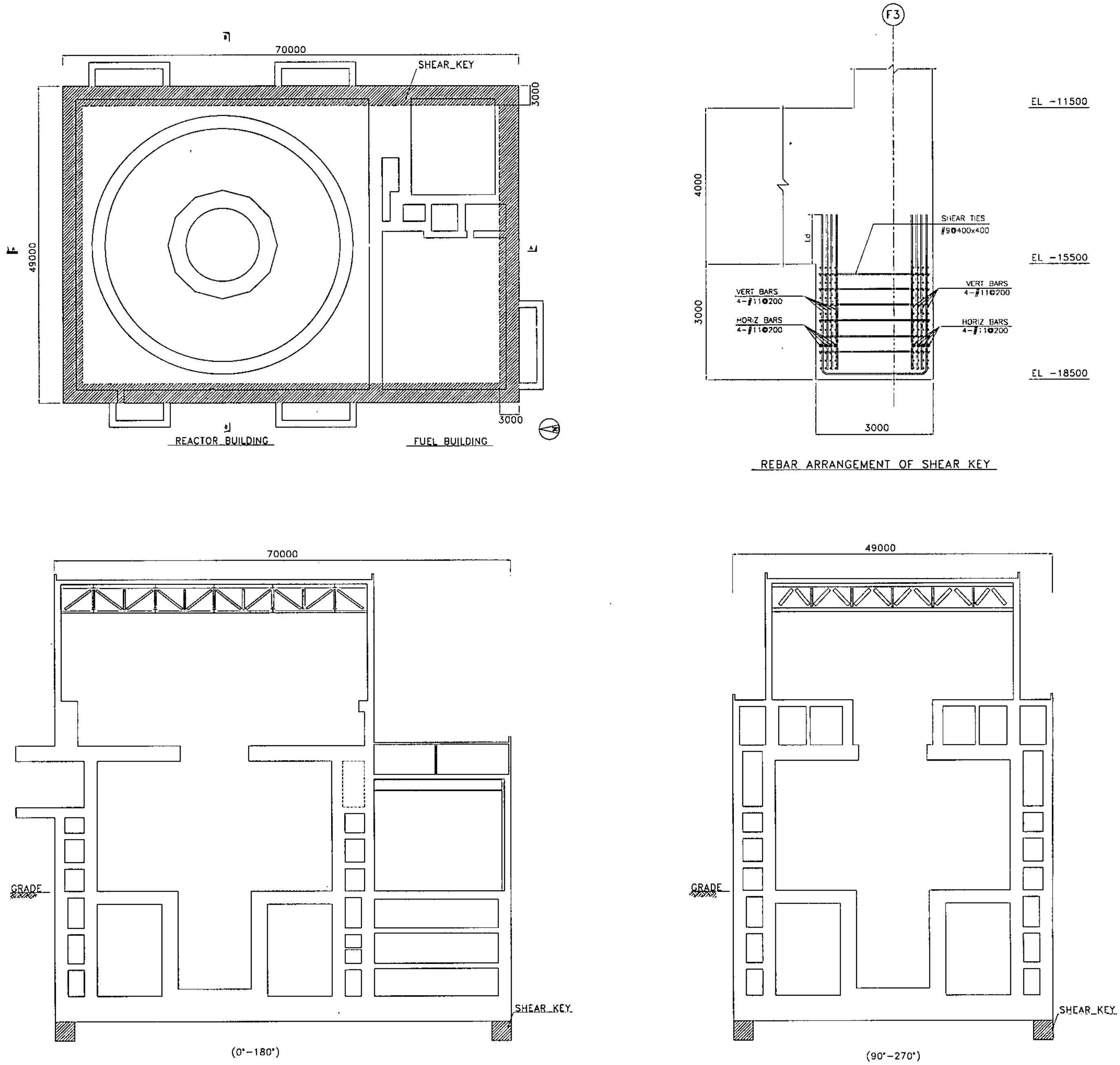


Figure 3.8-96(2) Shear Key Configuration for the RB/FB Structure

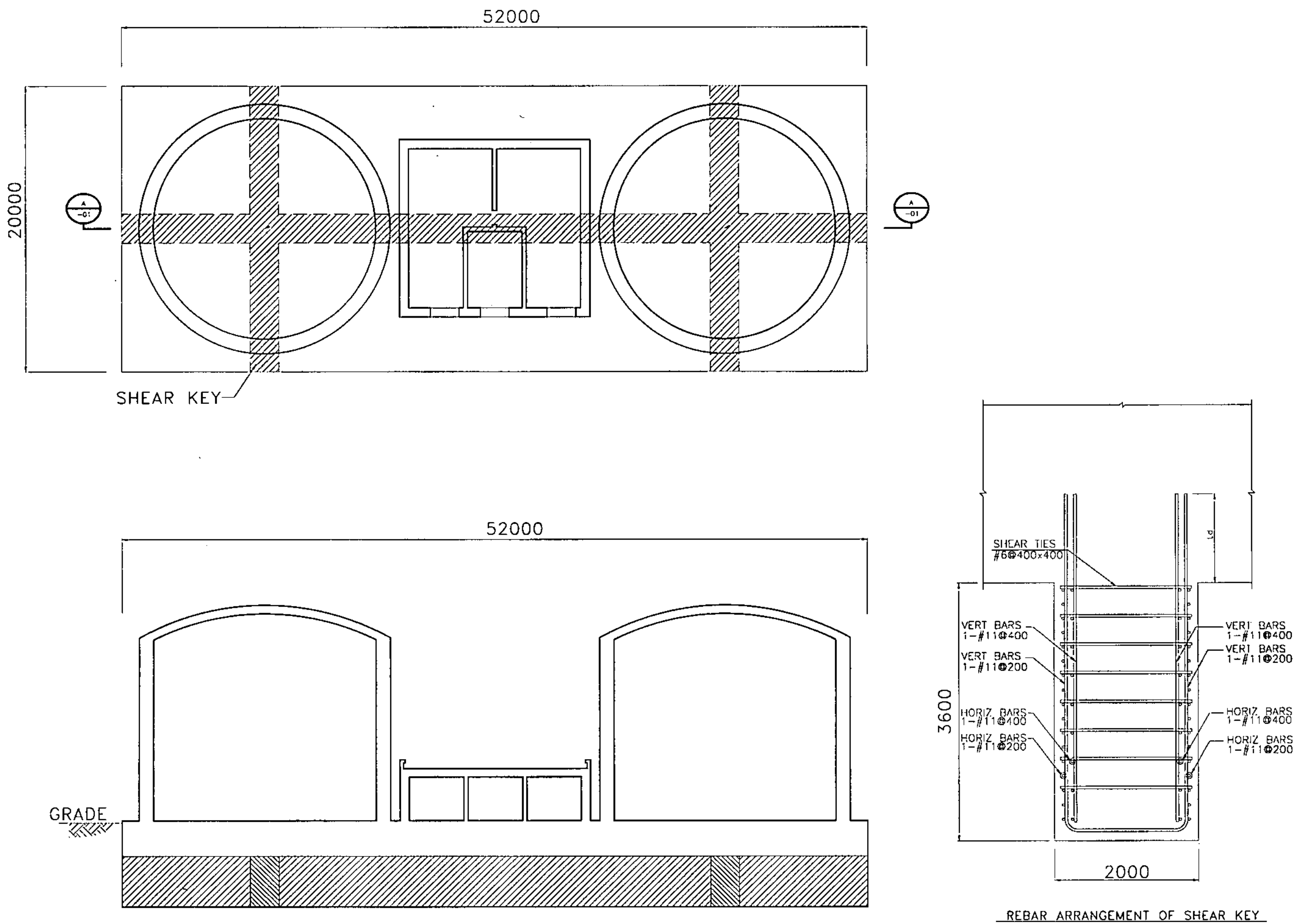


Figure 3.8-96(3) Shear Key Configuration for the FWSC Structure

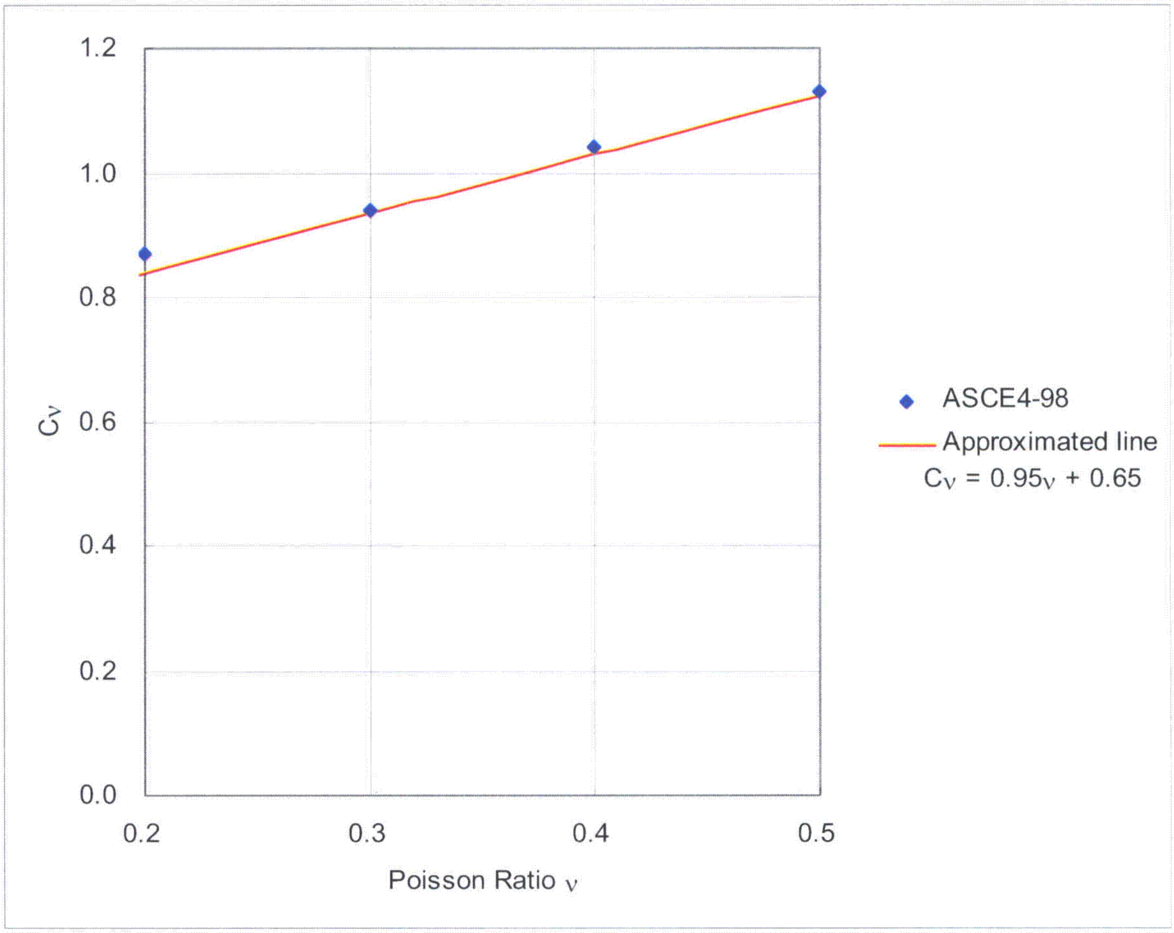


Figure 3.8-96(4) Coefficient as a Function of Poisson's Ratio

DCD Impact

Markups of DCD Tier 2 Subsections 3.8.5.5, 3.8.6.5 and 3G.1.5.5, Tables 2.0-1, 3G.1-57 and 3G.2-26 and Figures 3G.1-1, 3G.1-6, 3G.1-7 and 3G.4-1 were provided to the NRC in MFN 09-449, dated 7/1/09.

NRC RAI 3.8-96, Supplement 5, Revision 1

This RAI was previously sent as part of RAI Letter 368 and is been revised to add an item in bold below:

Please provide additional information against the items listed below:

Item C response is not adequate because it discusses the calculation for rotation of the foundation and concluded that the resulting maximum deformation from seismic loading is very small, and thus no passive pressures are developed. However, the evaluation of sliding stability in a number of locations requires the use of passive pressure on shear keys in order to satisfy the sliding stability safety factor. An example is page 35 of 42 of the RAI response which utilizes in equation (5) the term (kp – ka), where kp and ka are defined as full passive and active pressures respectively. It is not clear how the shear keys develop full passive pressure and still "ensure a non-slide condition." Also, please clarify the rationale for considering full passive pressure on the shear keys (F_r') and wall design lateral pressure (F_r) on the embedded wall in the sliding evaluation.

Item D response refers to a sliding evaluation approach presented at the end of the supplemental response. For all three structures (RB/FB, CB, and FWSC), GEH is requested to address the following items for this sliding evaluation:

(a) The lateral resistance pressure along the foundation wall and basemat (Fr) perpendicular to the direction of motion is defined as the wall design lateral pressure, which consists of the at-rest static earth pressure and the dynamic seismic earth pressure from SASSI analysis and the ASCE 4-98 elastic solution. Since during a seismic event seismic forces will be acting on both sides of a given building, please clarify how this was considered in the evaluation of sliding stability. Also, please describe the criterion for selecting the dynamic seismic earth pressure calculated from the SASSI analysis and ASCE 4-98 elastic solution in the sliding evaluation. Was the same criterion used for both design of wall as well as for computing sliding resistance?

(b) With the troughs added to the bottom of the mud mat and the use of shear keys beneath the basemat, the governing sliding interface may now be a horizontal plane in the soil at the elevation corresponding to the bottom of the shear keys. At this elevation there would no longer be any lateral resistance contribution from the surcharge of the building acting on the shear keys when calculating Fus' and Fr'. Explain whether another calculation was performed to determine the sliding factor of safety at the elevation of the bottom of the shear keys and describe the results of that evaluation.

Item E response stated that the vertical edges of the basemat do not use waterproofing membrane and instead sprayed with crystalline waterproofing material to ensure that the 0.7 coefficient of friction is achieved. It is not clear why surface preparation similar to the basemat was not necessary for the vertical edges of the shear keys and the basemat to ensure failure surface can only occur in the soil.

Item G response does not adequately address the question raised in the RAI. Based on the prior submittal of information, it was indicated that the crystalline material is effective up to 0.4 mm size cracks. Please describe if the crystalline material is used both in the *mudmat concrete mix and also applied to the top surface of the mudmat. Also please explain how wide the contraction joints will be and how the contraction joints will be ensured to be waterproof.*

Item H response identified the type, thickness and number of layers of the waterproofing membrane material applied to the exterior walls. The response also indicated that the waterproofing system is a sheet-applied barrier material as described in ACI 515.1R-79 (revised 1985). Since the use of the waterproofing membrane material is in accordance with the industry standard ACI 515.1R-79 (revised 1985), this item is technically acceptable. However, this information needs to be placed in the appropriate sections of the DCD.

Item I response provided the proposed markups to the various sections of the DCD. Since there are several other items still unresolved as discussed above, this Item I is still unresolved. GEH is requested to incorporate any additional revisions to the DCD resulting from the resolution of the other items.

New Item J: As a result of the staff's review of the RAI 3.8-96 S04 response, it was not clear from the revised DCD Tier 2 Table 2.0-1 and the corresponding DCD Tier 1 Table 5, if all important soil parameters (including shear wave velocity) for the backfill and surrounding soil (beyond the backfill and beneath the foundation), that were relied upon for the various seismic evaluations (i.e., stability, soil bearing, building design, as well as SSI analyses), were included in the DCD tables. GEH is requested to ensure that all important soil parameters relied upon for the various analyses are appropriately reflected in the DCD as required site parameters.

GEH Response (Revision 1)

C) The revised sliding detailed evaluation shown at the end of this supplemental response includes passive pressure on the embedded exterior walls and the shear keys and demonstrates that a minimum factor of safety against sliding of 1.1 is achieved. To maintain consistency in the assumptions for passive resistance, a "wall capacity" passive pressure is determined to be the pressure under which the embedded exterior wall is stressed, under SSE in combination with other applicable loads, to the ACI 349-01 allowable limits while maintaining the sliding factor of safety of 1.1 minimum. The distribution of the wall capacity pressure follows the triangular pattern of the passive pressure and is extended to the depth of shear keys when used as shown in Figures 3.8-96(5) and 3.8-96(7). This calculation is made for RB/FB and CB. The FWSC has no embedded walls and its passive resistance is provided by the shear keys sized to resist full passive pressure less active.

D)

(a) The lateral resistance pressure along the embedded exterior wall and basemat (F_r) is redefined to be the "wall capacity" passive pressure described in Item C above. In addition, active pressure is applied on the driving side. The magnitude of the active/passive pressure is adjusted to maintain the same proportion of the "wall capacity" passive pressure relative to the full passive pressure. Furthermore, the base shear $F_v(t)$ and base vertical force component $V_z(t)$ are replaced by respective overall force time histories of the embedded system as the summation of soil reaction forces at all interface nodes with soil at the base and side walls in the SASSI model for all soil conditions. Two separate cases, NS $F_v(t)$ together with vertical $V_z(t)$ and EW $F_v(t)$ together with vertical $V_z(t)$, are considered for each soil condition. The revised sliding detailed evaluation is shown at the end of this supplemental response.

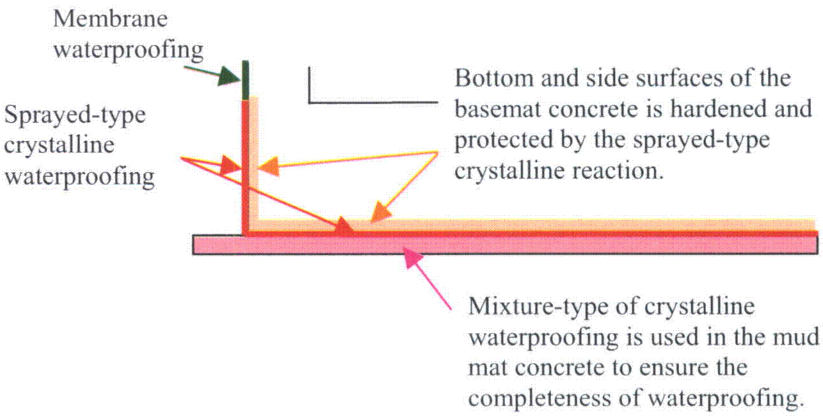
(b) The sliding calculation is redefined to include passive pressure on the embedded exterior walls and the shear keys to determine the sliding factor of safety at the elevation of the bottom of the shear keys for the RB/FB. The lateral resistance contribution from the surcharge of the building acting on the shear keys is eliminated from the Equations 4 and 5 for F_{us}' and F_r' in GEH's response to NRC RAI 3.8-96 S04 (MFN 09-449, dated 7/1/09).

When the F_{us}' and F_r' inside the shear keys are eliminated, then the weight of the soil above the sliding interface plane can be considered for calculating the base friction forces.

The various driving and resisting forces are illustrated in Figure 3.8-96(5). The revised sliding evaluation is shown at the end of this supplemental response.

E) To eliminate the need for surface preparation on the vertical surfaces of the basemat, the skin friction resistance from the vertical face of the basemat and shear keys uses a reduced friction coefficient of 0.5. The revised sliding detailed evaluation is shown at the end of this supplemental response.

G) The crystalline waterproofing compound is used in the mud mat concrete mix and is also applied to the top surface of the mud mat as stated in Item 10 of GEH's response to NRC RAI 3.8-96 S03 (MFN 06-407 S14, dated 2/20/09). Please see the illustration below.



For the vertical edges of the basemat, spray-type crystalline waterproofing compound will be applied in accordance with manufacturer application procedures.

Contraction joints are made after the mud mat concrete is poured to control cracks. The width and spacing of the contraction joints follow the common practice for pavements. The spray-type crystalline waterproofing compound applied on the top surface of the mud mat will fill up any cracks in the mud mat that have been formed. After application of the crystalline waterproofing compound, which has self-healing capability up to a 0.4 mm crack width, this waterproofing compound will be able to eliminate cracks in the mud mat concrete.

The above discussion will be included in Revision 7 of DCD Tier 2 Subsection 3.8.6.1.

H) The information provided in Item H of GEH's response to NRC RAI 3.8-96 S04 (MFN 09-449, dated 7/1/09) about the waterproofing membrane material applied to the embedded exterior walls will be placed in Revision 7 of DCD Tier 2 Table 1.9-22 and Subsection 3.8.6.1.

I) The revised sliding detailed evaluation and results obtained are shown below. DCD Tier 2 Table 1.9-22, Table 2.0-1, Subsection 3.8.5.5, Subsection 3.8.6.1, Subsection 3G.1.5.4.3.1, Subsection 3G.1.5.5, Tables 3G.1-50 through 3G.1-57e, Figures 3G.1-1, 3G.1-6, 3G.1-7 and 3G.1-47, Subsection 3G.2.5.5, Tables 3G.2-26, 3G.2-26a and 3G.2-26b, Subsection 3G.3.5.4.1, Tables 3G.3-13 through 3G.3-17, and Figures 3G.3-4 and 3G.3-5 will be revised or added in Revision 7 accordingly.

J) All important soil parameters relied upon for the various analyses are appropriately reflected in the DCD as required site parameters. For clarification, the following changes will be made in Revision 7 of the DCD:

- The first sentence of DCD Tier 1 Table 5.1-1, Note (3) will be revised to read, "This is the minimum shear wave velocity of the supporting foundation material and material surrounding the embedded walls associated with seismic strains for lower bound soil properties at minus one sigma from the mean".
- The following text will be added to the "Soil Properties" section of DCD Tier 1 Table 5.1-1 under "Angle of Internal Friction" for consistency with DCD Tier 2 Table 2.0-1: "(in-situ and backfill)".
- The requirements for backfill on sides of and underneath Seismic Category I structures will be added to the "Soil Properties" section of DCD Tier 1 Table 5.1-1 for consistency with DCD Tier 2 Table 2.0-1.
- The following text will be deleted from the "Soil Properties" section of DCD Tier 2 Table 2.0-1: "(not applicable if the fill material is concrete)".
- The minimum at-rest pressure coefficient information will be deleted from the "Soil Properties" section of DCD Tier 2 Table 2.0-1.

- The minimum soil density property of 1900 kg/m^3 (119 lbf/ft^3) will be changed to 2000 kg/m^3 (125 lbf/ft^3) in DCD Tier 2 Table 2.0-1 for consistency with DCD Tier 2 Table 3A.3-1 and is reflected in the revised sliding detailed evaluation shown at the end of this supplemental response.
- The first sentence of DCD Tier 2 Table 2.0-1, Note (8) will be revised to read, "This is the minimum shear wave velocity of the supporting foundation material and material surrounding the embedded walls associated with seismic strains for lower bound soil properties at minus one sigma from the mean".

The soil property for liquefaction potential under other than Seismic Category I structures in DCD Tier 2 Table 2.0-1 is not included in DCD Tier 1 Table 5.1-1 because this soil property is supplied as a COL action item; therefore, the information is not contained in DCD Tier 2.

Revised Detailed Evaluation

1. Soil Properties

The following soil properties are considered in the sliding evaluation. They are site parameter requirements for backfill on sides and underneath of Seismic Category I structures.

- Angle of internal friction
  $\phi = 35$ degree minimum
- Soil density
  $\gamma = 2000\ kg/m^3$ ($125\ lbf/ft^3$) minimum
- Product of at-rest soil pressure coefficient and density
  $k_o\gamma = 750\ kg/m^3$ ($47\ lbf/ft^3$) minimum

2. Sliding Evaluation

Time-consistent phasing between the SSE overall horizontal shear and vertical force is considered to compute the sliding factor of safety (*FS(t)*) as a function of time when combined with deadweight and upward buoyancy force.

(a) RB/FB structure

The FS is evaluated by taking the minimum values of the *FS(t)* time history calculated per the following equation. See Figure 3.8-96(5) for each force.

$$FS(t) = \frac{F_{ub}(t) + F_{us} + F_r + F_{us}' + F_r'}{F_v(t) + F_o} \qquad (1)$$

where,

$F_v(t)$: Overall force time histories of the embedded system as the summation of soil reaction forces at all interface nodes with soil at the base and side walls in the SASSI model.

F_o: Lateral soil force on RB due to TB surcharge load.

$F_{ub}(t)$: Friction resistance force of the sliding plane at the bottom of shear keys. Sliding potential at the basemat/mud mat and mud mat/soil interfaces is precluded through intentional roughening of the mud mat top surface and making troughs in the soil underneath the mud mat.

$$F_{ub}(t) = P\ tan\phi = (0.9D - B - V_z(t))\ tan\phi \qquad (2)$$

where D: Dead weight
$V_z(t)$: Vertical seismic force time history
B: Buoyancy

F_{us}: Skin friction resistance force provided by basemat side parallel to the direction of motion.

$$F_{us} = \mu P_o \qquad (3)$$

where,

μ: Skin friction coefficient of vertical sides of basemat parallel to the direction of motion (=0.5).

$P_o = k_o \gamma L(H_2^2 - H_1^2)/2$:
At-rest soil force normal to the basemat vertical surface neglecting surcharge term and water pressure term.

L: Skin friction length of both sides of basemat parallel to the direction of motion.

H_1, H_2: Embedment depths at the top and bottom of basemat, respectively.

F_{us}': Skin friction resistance force provided by the outside vertical surface of shear key parallel to the direction of motion.

$F_{us}' = \mu P_o'$ ....(4)

where,

μ: Skin friction coefficient of outside vertical surface of shear key parallel to the direction of motion (=0.5).

$P_o' = k_o \gamma L'(H_3^2 - H_2^2)/2$:
At-rest soil force normal to the shear key vertical surface.

L': Skin friction length of outside surfaces of shear key parallel to the direction of motion.

H_2, H_3: Embedment depths at the top and bottom of shear key, respectively.

F_r: Lateral resistance pressure along the embedded exterior wall and basemat opposite to the direction of motion.

$F_r = \beta (k_p - k_a) \gamma L H_2^2 /2$ ....(5)

where,

$k_p = (1+\sin\phi)/(1-\sin\phi)$: Rankine's passive pressure coefficient

$k_a = (1-\sin\phi)/(1+\sin\phi)$: Rankine's active pressure coefficient

L: Horizontal length of building opposite to the direction of motion.

H_2: Embedment depth at the bottom of basemat.

β: Reduction factor of full passive/active pressure.
It is equal to 0.81 in NS direction and 0.52 in EW direction.

The reduction factor, β, is determined to maintain the sliding factor of safety to be 1.1 minimum while the "wall capacity" passive pressure will not exceed the code allowable stresses. This check is performed for the embedded exterior walls under SSE in combination with other applicable loads according to ACI 349-01 allowable limits. The distribution of the reduced passive pressure follows the triangular pattern of the passive pressure.

F_r': Lateral resistance pressure along shear key opposite to the direction of motion.

$F_r' = \beta (k_p - k_a) \gamma L'(H_3^2 - H_2^2)/2$ ....(6)

where,

$k_p = (1+\sin\phi)/(1-\sin\phi)$: Rankine's passive pressure coefficient.

$k_a = (1-\sin\phi)/(1+\sin\phi)$: Rankine's active pressure coefficient.

L': Horizontal length of shear key opposite to the direction of motion.

H_2, H_3: Embedment depths at the top and bottom of shear key.
β: Reduction factor of full passive/active pressure, same as in equation (5).

The following are the calculated results of individual forces for the RB/FB for the governing FS case:

direction		NS	EW
governing site soil condition		HARD	RL-4
Time	(sec)	7.165	7.300
$F_v(t)$	(MN)	1187	1181
F_o	(MN)	128	0
$F_{ub}(t)$	(MN)	602	552
F_{us}	(MN)	37	26
F_{us}'	(MN)	45	26
F_r	(MN)	532	488
F_r'	(MN)	234	215
FS		1.10	1.11

It is found from the results of the "wall capacity" check that the current rebar arrangements of the exterior walls need to be revised as shown in Tables 3.8-96(8) and 3.8-96(12) for the RB and the FB, respectively. The stress check results for the revised rebar arrangements are shown in Tables 3.8-96(9) through 3.8-96(11) and Tables 3.8-96(13) through 3.8-96(14).

The shear key configuration also needs to be revised as shown in Figure 3.8-96(6). The reinforcement in the shear key is determined to resist passive pressure less the active pressure.

(b) CB structure

The *FS* is evaluated by taking the minimum values of the *FS(t)* time history calculated per the following equation. See Figure 3.8-96(7) for each force.

$$FS(t) = \frac{F_{ub}(t) + F_{us} + F_r}{F_v(t)} \quad \text{.......} (7)$$

where,

$F_v(t)$: Overall force time histories of the embedded system as the summation of soil reaction forces at all interface nodes with soil at the base and side walls in the SASSI model.

$F_{ub}(t)$: Friction resistance force provided by basemat bottom.

$$F_{ub}(t) = P\tan\phi = (0.9D - B - V_z(t))\tan\phi \quad \text{.......} (8)$$

where D: Dead weight
$V_z(t)$: Vertical seismic force time history
B: Buoyancy

F_{us}: Skin friction resistance force provided by basemat side parallel to the direction of motion.

$F_{us} = \mu P_o$ ....(9)

where,

μ: Skin friction coefficient of vertical sides of basemat parallel to the direction of motion (=0.5).

$P_o = k_o \gamma L(H_2^2 - H_1^2)/2$: At-rest soil force normal to the basemat vertical surface neglecting surcharge term and water pressure term.

L: Skin friction length of both sides of basemat parallel to the direction of motion.

H_1, H_2: Embedment depths at the top and bottom of basemat, respectively.

F_r: Lateral resistance pressure along the embedded exterior wall and basemat opposite to the direction of motion.

$F_r = \beta(k_p - k_a)\gamma L H_2^2/2$ ....(10)

where,

$k_p = (1+\sin\phi)/(1-\sin\phi)$: Rankine's passive pressure coefficient.

$k_a = (1-\sin\phi)/(1+\sin\phi)$: Rankine's active pressure coefficient.

L: Horizontal length of building opposite to the direction of motion.

H_2: Embedment depth at the bottom of basemat.

β: Reduction factor of full passive/active pressure. It is equal to 0.41 in NS direction and 0.36 in EW direction.

The reduction factor, β, is determined to maintain the sliding factor of safety to be 1.1 minimum while the "wall capacity" passive pressure will not exceed the code allowable stresses. This check is performed for the embedded exterior walls under SSE in combination with other applicable loads according to ACI 349-01 allowable limits. The distribution of the reduced passive pressure follows the triangular pattern of the passive pressure.

The following are the calculated results of individual forces for the CB for the governing FS case:

direction		NS	EW
governing site soil condition		CL-2	HARD
Time	(sec)	8.690	7.290
$F_v(t)$	(MN)	77	88
$F_{ub}(t)$	(MN)	4	10
F_{us}	(MN)	9	7
F_r	(MN)	73	81
FS		1.11	1.12

It is confirmed from the results of the "wall capacity" check that the current rebar arrangements of the exterior walls are adequate. The stress check results are shown in Tables 3.8-96(15) and 3.8-96(16).

(c) FWSC structure

The FS is evaluated by taking the minimum values of the FS(t) time history calculated per the following equation. See Figure 3.8-96(8) for each force.

$$FS(t) = \frac{F_{ub}(t) + F_{us} + F_r + F_{us}' + F_r'}{F_v(t)} \qquad \text{.......(11)}$$

where,

$F_v(t)$: Base shear time history at bottom of basemat.

$F_{ub}(t)$: Friction resistance force provided by basemat bottom.

$F_{ub}(t) = P\ tan\phi = (0.9D - B - V_z(t))\ tan\phi$ ....(12)

where D: Dead weight

$V_z(t)$: Vertical seismic force time history

B: Buoyancy

F_{us}: Skin friction resistance force provided by basemat side parallel to the direction of motion.

$F_{us} = \mu P_o$ ....(13)

where,

μ: Skin friction coefficient of vertical sides of basemat parallel to the direction of motion (=0.5).

$P_o = k_o \gamma L H_1^2 / 2$: At-rest soil force normal to the basemat vertical surface neglecting surcharge term and water pressure term.

L: Skin friction length of both sides of basemat parallel to the direction of motion.

H_1: Embedment depth at the bottom of basemat.

F_{us}': Skin friction resistance force provided by vertical shear key surfaces parallel to the direction of motion.

$F_{us}' = \mu P_o'$ ....(14)

where,

μ: Skin friction coefficient of vertical surface of shear key parallel to the direction of motion (=0.5).

$P_o' = k_o \gamma L'(H_2^2 - H_1^2) / 2 + k_o q L'(H_2 - H_1)$: At-rest soil force normal to the shear key vertical surface.

q: Surcharge load of FWSC.

L': Skin friction length of both sides of shear key parallel to the direction of motion.

H_1, H_2: Embedment depths at the top and bottom of shear key, respectively.

F_r: Lateral resistance pressure along the basemat opposite to the direction of motion.

$F_r = \beta\,(k_p\text{-}k_a)\ \gamma L H_1^2\ /2$ .... (15)

where,

- $k_p = (1+sin\phi)/(1-sin\phi)$: Rankine's passive pressure coefficient.
- $k_a = (1-sin\phi)/(1+sin\phi)$: Rankine's active pressure coefficient.
- L: Horizontal length of basemat opposite to the direction of motion.
- H_1: Embedment depth at the bottom of basemat.
- β: Reduction factor of full passive/active pressure. It is equal to 0.98 in NS direction and 0.71 in EW direction.

The reduction factor, β, is determined to maintain the sliding factor of safety to be 1.1 minimum.

F_r': Lateral resistance pressure along shear key opposite to the direction of motion.

$F_r' = \beta\,\{(k_p\text{-}k_a)\ \gamma L'(H_2^2\text{-}H_1^2)\ /2+\ (k_p\text{-}k_a)qL'(H_2\text{-}H_1)\}$ .... (16)

where,

- $k_p = (1+sin\phi)/(1-sin\phi)$: Rankine's passive pressure coefficient.
- $k_a = (1-sin\phi)/(1+sin\phi)$: Rankine's active pressure coefficient.
- q: Surcharge load of FWSC.
- L': Horizontal length of shear key opposite to the direction of motion.
- H_1, H_2: Embedment depths at the top and bottom of shear key, respectively.
- β: Reduction factor of full passive/active pressure, same as in equation (15).

The following are the calculated results of individual forces for the FWSC for the governing FS case:

direction		NS	EW
governing site soil condition		FL-2	HARD
Time	(sec)	7.165	7.135
$F_v(t)$	(MN)	104	91
$F_{ub}(t)$	(MN)	41	23
F_{us}	(MN)	1	0
F_{us}'	(MN)	8	6
F_r	(MN)	9	16
F_r'	(MN)	56	55
FS		1.10	1.11

The current shear key configuration is adequate. The current reinforcement in the shear key is adequate to resist passive pressure less the active pressure.

3. Summary of Calculated FS

The calculated FS for the RB/FB, CB and FWSC for all site cases are summarized in Table 3.8-96(17).

Table 3.8-96(8) Sectional Thicknesses and Rebar Ratios of RB External Walls

Location	*Element ID*	*Thickness (m)*	*Primary Reinforcement*					*Shear Tie*	
			Position	*Horizontal*		*Vertical*			
				Arrangement	*Ratio (%)*	*Arrangement*	*Ratio (%)*	*Arrangement*	*Ratio (%)*
21 Exterior Wall @ EL-11.50 to -10.50m	*30010 30020*	*2.0*	*Inside*	*1-#11@100 +3-#11@200*	*1.258*	*2-#11@100 +2-#11@200*	*1.510*	*#6@200x200*	*0.710*
			Outside	*2-#11@100 +2-#11@200*	*1.510*	*3-#11@100 +1-#11@200*	*1.761*		
	40001 40011	*2.0*	*Inside*	*1-#11@100 +3-#11@200*	*1.258*	*2-#11@100 +2-#11@200*	*1.510*	*#6@200x200*	*0. 710*
			Outside	*2-#11@100 +2-#11@200*	*1.510*	*2-#11@100 +2-#11@200*	*1.510*		

Table 3.8-96(9) Rebar and Concrete Stresses of RB External Walls: Selected Load Combination RB-9a

Location	Element ID	Concrete Stress (MPa)		Primary Reinforcement Stress (MPa)				
		Calculated	Allowable	Calculated				Allowable
				Direction 1*		Direction 2*		
				In/Top	Out/Bottom	In/Top	Out/Bottom	
21 Exterior Wall @ EL-11.50 to -10.50m	30010	-7.6	-29.3	147.4	128.8	282.1	90.1	372.2
	30020	-5.6	-29.3	15.6	60.2	-9.0	74.5	372.2
	40001	-6.8	-29.3	36.8	40.2	22.5	82.5	372.2
	40011	-9.0	-29.3	124.1	125.5	291.7	104.1	372.2

Note: *Negative value means compression.*
*Note *:* *Direction 1 is Horizontal. Direction 2 is Vertical.*

Table 3.8-96(10) Rebar and Concrete Stresses of RB External Walls: Selected Load Combination RB-9b

Location	Element ID	Concrete Stress (MPa)		Primary Reinforcement Stress (MPa)				
		Calculated	Allowable	Calculated				Allowable
				Direction 1*		Direction 2*		
				In/Top	Out/Bottom	In/Top	Out/Bottom	
21 Exterior Wall @ EL-11.50 to -10.50m	30010	-5.5	-29.3	250.9	117.9	365.7	14.4	372.2
	30020	-5.8	-29.3	11.1	58.2	-9.1	65.3	372.2
	40001	-6.5	-29.3	8.3	64.8	6.5	93.3	372.2
	40011	-7.1	-29.3	235.7	112.3	362.7	59.8	372.2

Note: *Negative value means compression.*
*Note *:* *Direction 1 is Horizontal. Direction 2 is Vertical.*

Table 3.8-96(11) Transverse Shear of RB External Walls

Location	Element ID	Load ID	d (m)	pv (%)	Shear Force (MN/m)				Vu/ϕVn
					Vu	Vc	Vs	ϕVn	
21 Exterior Wall @ EL-11.50 to -10.50m	30010	RB-9a	1.69	0.710	2.29	0.07	4.97	4.29	0.533
	30020	RB-9a	1.71	0.710	0.76	1.08	5.02	5.18	0.146
	40001	RB-9a	1.71	0.710	1.19	1.03	5.03	5.15	0.230
	40011	RB-9a	1.69	0.710	2.98	0.29	4.97	4.47	0.667

Table 3.8-96(12) Sectional Thicknesses and Rebar Ratios of FB External Walls

Location	Element ID	Thickness (m)	Primary Reinforcement					Shear Tie	
			Position	Horizontal		Vertical			
				Arrangement	Ratio (%)	Arrangement	Ratio (%)	Arrangement	Ratio (%)
1 Exterior Wall and Pool Wall Bottom	*60011*	*2.0*	*Inside*	*3-#11@200*	*0.755*	*1-#11@100 +3-#11@200*	*1.258*	*#6@200x200*	*0.710*
			Outside	*1-#11@100 +3-#11@200*	*1.258*	*2-#11@100 +2-#11@200*	*1.510*		
	60219	*3.6*	*Inside*	*6-#11@200*	*0.839*	*6-#11@200*	*0.839*	*#6@200x200*	*0.710*
			Outside	*1-#11@100 +7-#11@200*	*1.258*	*1-#11@100 +7-#11@200*	*1.258*		
	70201 70204	*2.0*	*Inside*	*3-#11@100 +1-#11@200*	*1.761*	*3-#11@100 +1-#11@200*	*1.761*	*#6@200x200*	*0.710*
			Outside	*3-#11@100 +1-#11@200*	*1.761*	*5-#11@100*	*2.516*		
4 Spent Fuel Pool Wall @ EL-5.10 to -3.30m	*60819*	*3.6*	*Inside*	*6-#11@200*	*0.839*	*6-#11@200*	*0.839*	*#6@200x200*	*0.710*
			Outside	*1-#11@100 +7-#11@200*	*1.258*	*1-#11@100 +7-#11@200*	*1.258*		
	70801 70804	*2.0*	*Inside*	*3-#11@100 +1-#11@200*	*1.761*	*3-#11@100 +1-#11@200*	*1.761*	*#6@200x200*	*0.710*
			Outside	*3-#11@100 +1-#11@200*	*1.761*	*5-#11@100*	*2.516*		

Table 3.8-96(13) Rebar and Concrete Stresses of FB External Walls: Selected Load Combination FB-9

Location	Element ID	Concrete Stress (MPa)		Primary Reinforcement Stress (MPa)				
		Calculated	Allowable	Calculated				Allowable
				Direction 1*		Direction 2*		
				In/Top	Out/Bottom	In/Top	Out/Bottom	
1 Exterior Wall and Pool Wall Bottom	60011	-12.8	-29.3	264.4	158.3	303.6	96.4	372.2
	60219	-27.8	-28.5	-36.7	319.5	-96.6	263.1	366.4
	70201	-22.7	-28.3	-21.3	341.8	-41.6	295.7	364.6
	70204	-20.5	-28.3	-34.2	354.5	-56.4	363.9	364.6
4 Spent Fuel Pool Wall @ EL-5.10 to -3.30m	60819	-16.1	-28.5	-47.4	169.0	-53.7	179.8	366.4
	70801	-19.6	-28.3	-15.5	329.6	-36.0	250.0	364.6
	70804	-20.6	-28.3	-46.5	216.9	-57.8	215.5	364.6

Note: *Negative value means compression.*
*Note *:* *Direction 1 is Horizontal. Direction 2 is Vertical.*

Table 3.8-96(14) Transverse Shear of FB External Walls

Location	Element ID	Load ID	d (m)	pv (%)	Shear Force (MN/m)				Vu/ϕVn
					Vu	Vc	Vs	ϕVn	
1 Exterior Wall and Pool Wall Bottom	60011	FB-9	1.69	0.710	1.66	1.00	4.97	5.07	0.328
	60219	FB-9	3.05	0.710	7.51	3.93	8.96	10.95	0.686
	70201	FB-9	1.62	0.710	1.37	0.00	4.75	4.04	0.339
	70204	FB-9	1.59	0.710	2.04	0.09	4.68	4.05	0.504
4 Spent Fuel Pool Wall @ EL-5.10 to -3.30r	60819	FB-8	3.05	0.710	2.07	3.26	8.96	10.39	0.199
	70801	FB-9	1.71	0.710	2.12	1.45	5.03	5.51	0.385
	70804	FB-9	1.61	0.710	0.65	2.09	4.72	5.79	0.112

Table 3.8-96(15) Rebar and Concrete Stresses of CB External Walls: Selected Load Combination CB-9

Location	Element ID	Concrete Stress (MPa)		Primary Reinforcement Stress (MPa)				
		Calculated	Allowable	Calculated				Allowable
				Horizontal direction		Vertical direction		
				Inside	Outside	Inside	Outside	
Wall EL-7.4m ~EL-2.0m	6007	-11.4	-29.3	148.9	252.2	105.4	277.3	372.2
	4006	-13.9		69.7	190.8	143.4	241.0	
	4010	-5.1		95.5	144.2	60.6	225.4	
Wall EL-2.0m ~EL4.65m	6043	-9.7	-29.3	119.7	202.1	-12.9	85.9	372.2
	4036	-6.3		95.0	131.5	154.3	147.7	
	4040	-6.6		129.7	159.5	190.7	190.1	

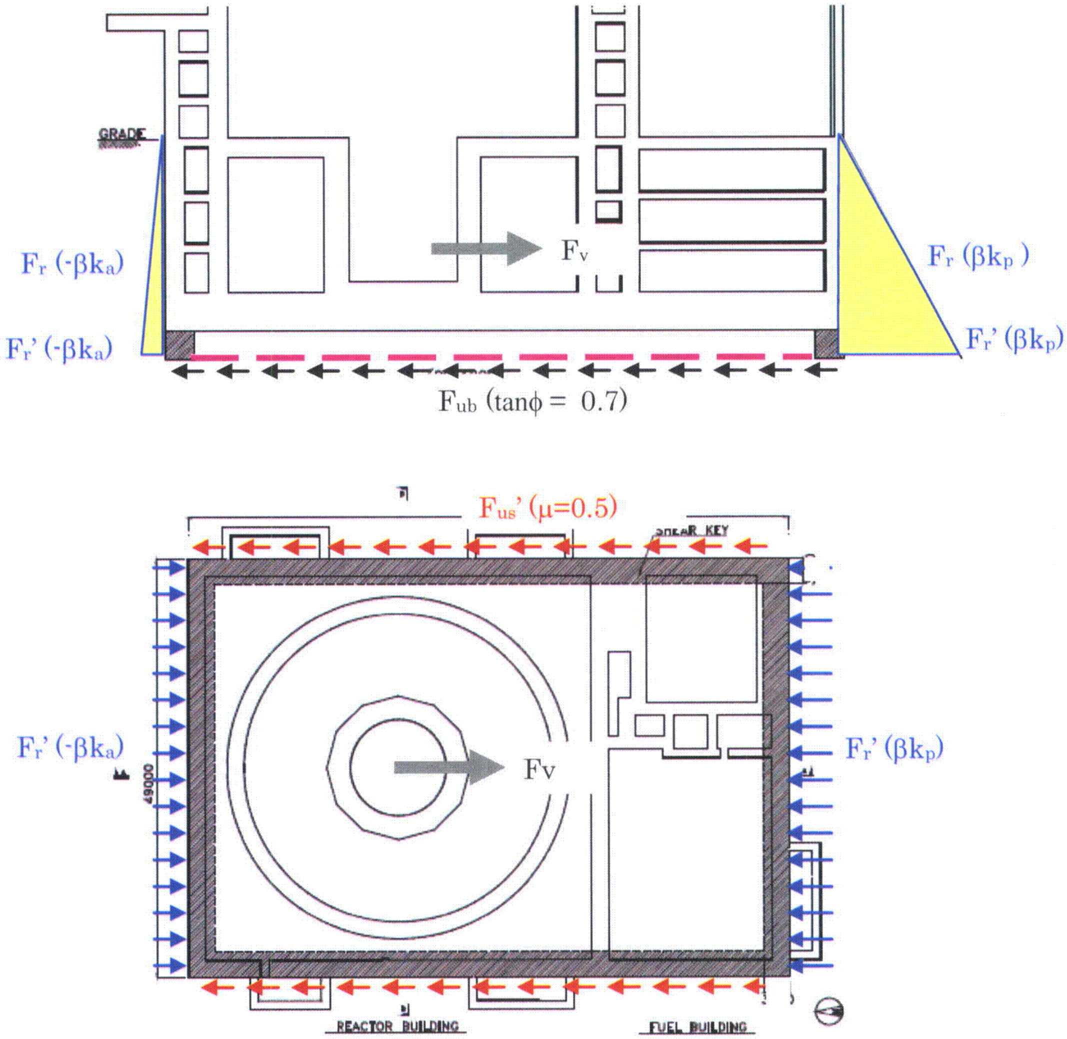
Note: Negative value means compression.

Table 3.8-96(16) Transverse Shear of CB External Walls

Location	Element ID	Load ID	d (m)	ρ_w (%)	ρ_v (%)	Shear Forces (MN/m)				Vu/ϕVn
						Vu	Vc	Vs	ϕVn	
Wall EL-7.4m ~EL-2.0m	6007	CB-9	0.71	1.42	0.36	0.14	0.07	1.04	0.95	0.14
	4006	CB-9	0.67	1.50	0.36	0.57	0.20	0.99	1.01	0.56
	4010	CB-9	0.68	1.49	0.36	0.68	0.48	0.99	1.26	0.54
Wall EL-2.0m ~EL4.65m	6043	CB-9	0.67	1.50	0.36	0.22	0.51	0.99	1.27	0.17
	4036	CB-9	0.67	1.50	0.71	0.58	0.25	1.98	1.89	0.31
	4040	CB-9	0.69	1.46	0.36	0.28	0.12	1.01	0.96	0.29

Table 3.8-96(17) Summary of Factor of Safety for Sliding

	L-1		L-2		L-3		L-4		SOFT		MEDIUM		HARD		Minimum FS
	NS dir.	EW dir.	NS dir.	EW dir.	NS dir.	EW dir.	NS dir.	EW dir.	NS dir.	EW dir.	NS dir.	EW dir.	NS dir.	EW dir.	
RB/FB	1.78	2.43	1.27	1.16	1.95	3.12	1.39	1.11	2.13	3.33	1.38	1.26	1.10	1.33	1.10
CB	2.02	1.89	1.11	1.33	2.11	1.94	1.14	1.39	2.05	1.98	1.33	1.28	1.12	1.12	1.11
FWSC	1.28	1.27	1.10	1.28	1.29	1.44	1.12	1.20	1.29	1.43	1.28	1.29	1.12	1.11	1.10



Note: This is the plan view at the bottom of shear keys. The corresponding forces are Fr on the embedded exterior walls and Fr and Fus on the basemat.

Figure 3.8-96(5) Forces for RB/FB Sliding Evaluation

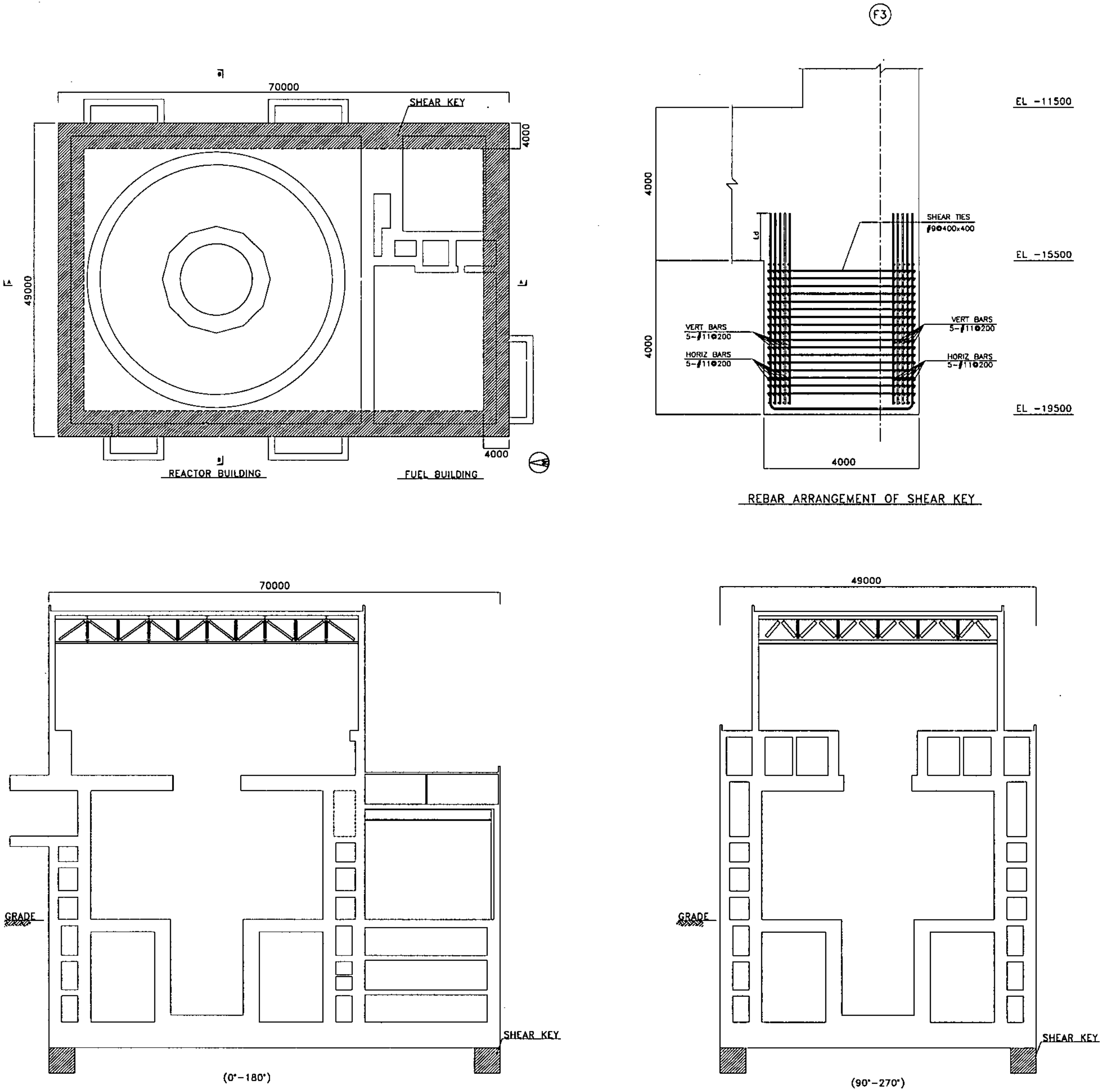
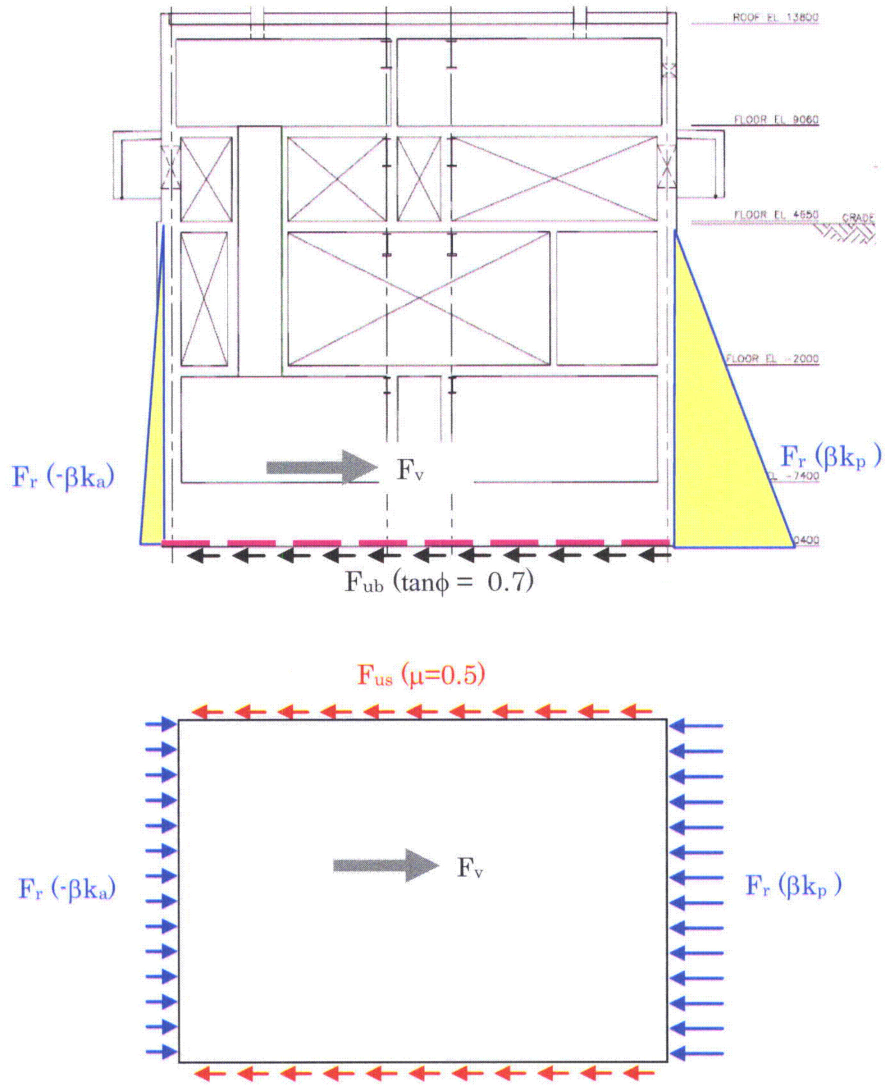
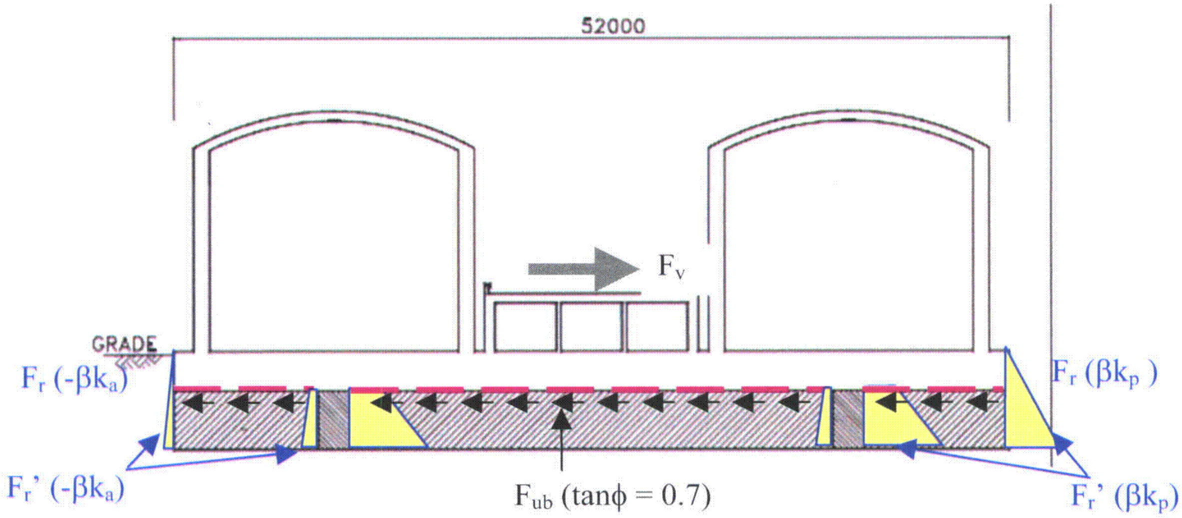


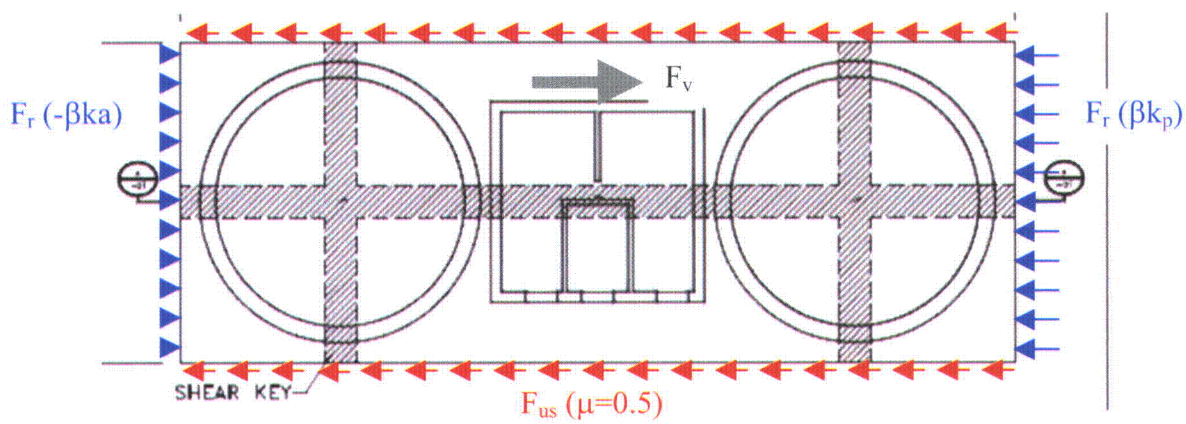
Figure 3.8-96(6) Revised Shear Key Configuration of RB/FB



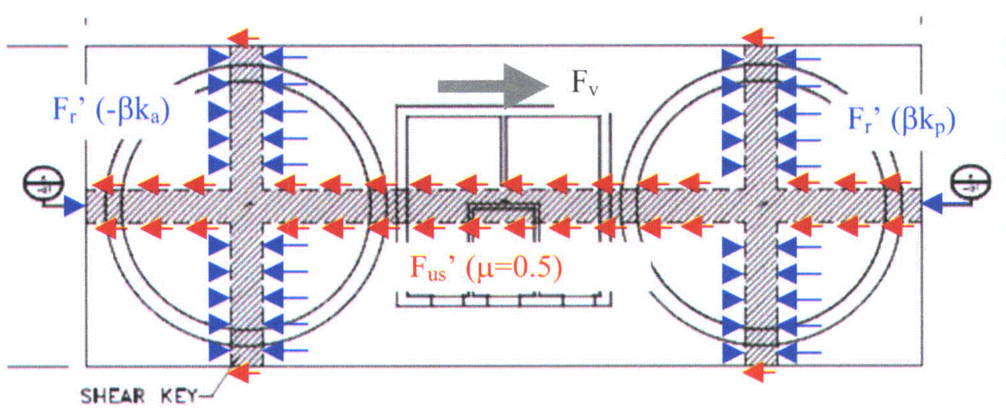
Note: This is the plan view at the basemat. The corresponding forces are Fr only on the embedded exterior walls.

Figure 3.8-96(7) Forces for CB Sliding Evaluation





Note: This is the plan view on the basemat.



Note: This is the plan view on the shear keys.

Figure 3.8-96(8) Forces for FWSC Sliding Evaluation

DCD Impact

DCD Tier 1 Table 5.1-1 will be revised in Revision 7 as noted in the attached markup.

DCD Tier 2 Subsection 3.8.6.1 will be revised in Revision 7 as noted in the attached markup.

A markup of Note (3) of Revision 7 of DCD Tier 1 Table 5.1-1 was provided to the NRC in MFN 09-772, dated 12/12/09~~will be revised in Revision 7 as noted in the attached markup~~.

Markups of DCD Tier 2 Table 1.9-22, Table 2.0-1, Subsection 3.8.5.5, Subsection 3.8.6.1, Subsection 3G.1.5.4.3.1, Subsection 3G.1.5.5, Tables 3G.1-50 through 3G.1-57e, Figures 3G.1-1, 3G.1-6, 3G.1-7 and 3G.1-47, Subsection 3G.2.5.5, Tables 3G.2-26, 3G.2-26a and 3G.2-26b, Subsection 3G.3.5.4.1, Tables 3G.3-13 through 3G.3-17, and Figures 3G.3-4 and 3G.3-5 were provided to the NRC in MFN 09-772, dated 12/12/09~~will be revised or added in Revision 7 as noted in the attached markups~~.

Enclosure 2

MFN 09-772 Revision 1

Revised Response to Portion of NRC Request for

Additional Information Letter No. 368

Related to ESBWR Design Certification Application

Revised DCD Tier 1 and Tier 2 Markups for

RAI Number 3.8-96 S05 Revision 1

Table 5.1-1
Envelope of ESBWR Standard Plant Site Parameters (continued)

Soil Properties: (6)	- Minimum Static Bearing Capacity (2): Greater than or equal to the maximum static bearing demand multiplied by a factor of safety appropriate for the design load combination. Maximum Static Bearing Demand: ~~(2)~~ Reactor/Fuel Building: 699 kPa (14,600 lbf/ft^2) Control Building: 292 kPa (6,100 lbf/ft^2) Fire Water Service Complex: 165 kPa (3,450 lbf/ft^2) - Minimum Dynamic Bearing Capacity (2): Greater than or equal to the maximum dynamic bearing demand multiplied by a factor of safety appropriate for the design load combination. Maximum Dynamic Bearing Demand (SSE + Static): ~~(2)~~ **Reactor/Fuel Building:** Soft: 1100 kPa (23,000 lbf/ft^2) Medium: 2700 kPa (56,400 lbf/ft^2) Hard: 1100 kPa (23,000 lbf/ft^2) **Control Building:** Soft: 500 kPa (10,500 lbf/ft^2) Medium: 2200 kPa (46,000 lbf/ft^2) Hard: 420 kPa (8,800 lbf/ft^2) **Firewater Service Complex (FWSC):** Soft: 460 kPa (9,600 lbf/ft^2) Medium: 690 kPa (14,400 lbf/ft^2) Hard: 1200 kPa (25,100 lbf/ft^2) - Minimum Shear Wave Velocity: (3) 300 m/s (1000 ft/s) - Liquefaction Potential: Seismic Category I Structures: None under footprint of Seismic Category I structures resulting from site-specific SSE. - Angle of Internal Friction (in-situ and backfill): ≥ 35 degrees - Backfill on sides of and underneath Seismic Category I structures Product of peak ground acceleration α (in g), Poisson's ratio ν and density γ: $\alpha(0.95\nu+0.65)\gamma$: 1220 kg/m^3 (76 lbf/ft^3) maximum Product of at-rest pressure coefficient k_0 and density: $k_0\gamma$: 750 kg/m^3 (47 lbf/ft^3) minimum Soil density: γ: 2000 kg/m^3 (125 lbf/ft^3) minimum
Seismology:	- SSE Horizontal Ground Response Spectra: (4) See Figure 5.1-1 - SSE Vertical Ground Response Spectra: (4) See Figure 5.1-2

vertical SSE force acting upward. The total vertical load at the base takes into account the dead loads and buoyancy force.

The factor of safety against flotation is defined as:

$FS = F_{DL}/F_B$

Notations are as follows:

F_{DL} = Downward force due to dead load.

F_B = Upward force due to buoyancy.

Text sections that are bracketed and italicized with an asterisk following the brackets are designated as Tier 2. Prior NRC approval is required to change.

3.8.5.6 Materials, Quality Control, and Special Construction Techniques

[*The foundations of Seismic Category I structures are constructed of reinforced concrete using proven methods common to heavy industrial construction. For further discussion, see Subsection 3.8.1.6 for the containment foundation mat and Subsection 3.8.4.6 for the foundations of the other Seismic Category I structures.*]*

Text sections that are bracketed and italicized with an asterisk following the brackets are designated as Tier 2. Prior NRC approval is required to change.

3.8.5.7 Testing and In-Service Inspection Requirements

The foundations of Seismic Category I structures are monitored per NUREG-1801 and 10 CFR 50.65 as clarified in Section 1.5 of RG 1.160.

3.8.6 Special Topics

3.8.6.1 Foundation Waterproofing

[*The selected waterproofing material for the bottom of the basemat is a chemical crystalline powder that is added to the mud mat mixture forming a water proof barrier when cured. For the vertical edges of the basemat, spray-type crystalline waterproofing compound will be applied in accordance with manufacturer application procedures. No membrane waterproofing is used under the foundations in ESBWR.*]*

Contraction joints are made after the mud mat concrete is poured to control cracks. The width and spacing of the contraction joints follow the common practice for pavements. The spray-type crystalline waterproofing compound applied on the top surface of the mud mat will fill up cracks in the mud mat that have been formed. After application of the crystalline waterproofing compound, which has a self-healing capability up to a 0.4 mm crack width, this waterproofing compound will be able to eliminate cracks in the mud mat concrete.

The type of the waterproofing system applied to the exterior walls is sheet-applied barrier materials described in Section 4.2.1.4 of ACI 515.1R-79 (revised 1985) (e.g. non-vulcanized butyl rubber sheet). The minimum thickness of the waterproofing sheet is 2.0 mm. Two layers of sheets are applied to the exterior walls below grade.